# APPENDIX E

# GEOTECHNICAL INVESTIGATION AND INFILTRATION TESTING RESULTS

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# GEOTECHNICAL INVESTIGATION
# PROPOSED WAREHOUSE

26200 Enterprise Way
Lake Forest, California
for
Black Creek Group

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

April 7, 2021
(Revised November 18, 2021)

Black Creek Group
4675 MacArthur Court, Suite 625
Newport Beach, California 92660

Attention: Mr. Chris Sanford
Senior Vice President, Development

Project No.: 21G135-1R2

Subject: Geotechnical Investigation
Proposed Warehouse
26200 Enterprise Way
Lake Forest, California

Dear Mr. Sanford:

In accordance with your request, we have conducted a geotechnical investigation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

**SOUTHERN CALIFORNIA GEOTECHNICAL, INC.**

Gregory K. Mitchell, GE 2364
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Distribution: (1) Addressee

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# 1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

## Geotechnical Design Considerations

- The site is underlain by engineered fill soils and Capistrano formation sandstone bedrock. The bedrock generally consists of medium dense to very dense, poorly consolidated fine-grained sandstone. The fill soils possess relatively high strengths and favorable consolidation/collapse characteristics, indicate of engineered fill. We performed research at the City of Lake Forest in an attempt to obtain reports documenting the placement and compaction of any such fill soils. The City has no such records.
- The existing conditions will create a bedrock/fill transition within the proposed building area. It is also expected that the upper 3± feet of soils will be disturbed during demolition of the existing development. Based on these conditions, remedial grading will be necessary within the proposed building area to provide a pad suitable for support of the proposed structure.
- The existing pavements are in fair to good condition, and consideration may be given to reusing some of the existing pavements with the new development. However, the existing pavement thicknesses are not adequate to support any significant volume of truck traffic.
- The on-site soils possess a very low expansion potential.

## Site Preparation Recommendations

- Demolition of the existing development should include foundations, floor slabs, utilities, pavements and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of off-site. Concrete and asphalt debris may be crushed to a maximum 2-inch particle size, well-mixed with on-site soils, and reused in new structural fills.
- Initial site preparation should include stripping of any surficial vegetation within the landscaped planters that are demolished. The surficial vegetation, and any organic soils should be properly disposed of off-site.
- Remedial grading is recommended to be performed within the proposed building area in order to mitigate the bedrock/fill transitions, and to remove all soils disturbed during demolition of the existing building. The soils within the proposed building area should be overexcavated to a depth of 5 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevations, whichever is greater.
- The proposed foundation influence zones should be overexcavated to a depth of at least 3 feet below proposed foundation bearing grade.
- Additional overexcavation should be performed in the southeastern region of the building pad to mitigate the bedrock/fill transition. This area of the pad should be overexcavated to a depth of 5 feet below foundation bearing grade.
- Following completion of the overexcavation, the exposed soils should be scarified to a depth of at least 12 inches and moisture treated to 0 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

### Foundation Design Recommendations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 3,000 $lbs/ft^2$ maximum allowable soil bearing pressure.
- Reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

### Building Floor Slab Design Recommendations

- Conventional Slab-on-Grade: minimum 6 inches thick.
- Modulus of Subgrade Reaction: k = 150 psi/in.
- Reinforcement is not expected to be necessary for geotechnical considerations. The actual thickness and reinforcement of the floor slab should be determined by the structural engineer.

### Pavement Design Recommendations

| ASPHALT PAVEMENTS (R = 40) | | | | | |
|---|---|---|---|---|---|
| Materials | Thickness (inches) | | | | |
| | Parking Stalls (TI = 4.0) | Auto Drive Lanes (TI = 5.0) | Truck Traffic | | |
| | | | (TI = 6.0) | (TI = 7.0) | (TI = 8.0) |
| Asphalt Concrete | 3 | 3 | 3½ | 4 | 5 |
| Aggregate Base | 3 | 4 | 6 | 7 | 8 |
| Compacted Subgrade (90% minimum compaction) | 12 | 12 | 12 | 12 | 12 |

| PORTLAND CEMENT CONCRETE PAVEMENTS (R = 40) | | | | |
|---|---|---|---|---|
| Materials | Thickness (inches) | | | |
| | Automobile Parking and Drive Areas (TI = 5.0) | Truck Traffic | | |
| | | (TI =6.0) | (TI =7.0) | (TI =8.0) |
| PCC | 5 | 5 | 5½ | 6½ |
| Compacted Subgrade (95% minimum compaction) | 12 | 12 | 12 | 12 |

## 2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 21P170, dated February 25, 2021. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

# 3.0 SITE AND PROJECT DESCRIPTION

## 3.1 Site Conditions

The site is located at 26200 Enterprise Way in Lake Forest, California. The site is bounded to the north by Enterprise Way and existing commercial/industrial buildings, to the east and south by existing commercial/industrial developments, and to the west by Enterprise Court. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The site is an irregular-shaped parcel, 8.83± acres in size. The site is presently developed with one (1) commercial office building, with a first-floor footprint of 75,000± $ft^2$ in size, located in the central area of the site. The development is a two-story building of concrete tilt up construction. A series of Bloomenergy servers, are located on concrete pads along the southern property line, behind the building. The building is surrounded by asphaltic concrete pavements and limited areas of Portland cement concrete pavements. The pavements are in good condition with minor cracking throughout. Landscape planters are present throughout the site and possesses small shrubs, bushes and medium to large trees. The eastern and southern boundaries of the site possess north and west facing slopes. These slopes ascend 10 to 15 feet from the subject site to the adjacent property. A concrete-lined drainage swale is located near the midpoint of the slope. The remaining areas of the slopes are covered with dense vegetation.

Detailed topographic information was obtained from the preliminary grading and drainage plan, prepared by Kier + Wright. Based on this plan, the overall site slopes downward to the west at a gradient of 3± percent. As mentioned above, the eastern and southern boundaries of the site possess a north and west facing ascending slopes, with a gradient of approximately 3h:1v. The minimum site elevation is 690± feet mean sea level (msl), located at the west corner of the site. The maximum site elevation is 710± feet msl, located along the eastern property line.

## 3.2 Proposed Development

A preliminary site plan has been provided to our office by the client. Based on this plan, the site will be developed with one (1) new commercial industrial building, 168,467± $ft^2$ in size, located in the central region of the site. Dock-high doors will be constructed along a portion of the southern building wall. The proposed building is expected to be surrounded by asphaltic concrete pavements in the parking and drive areas, Portland cement concrete pavements in the loading dock areas, and concrete flatwork and landscaped planters throughout the site.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, typically supported on conventional shallow foundations with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 7 kips per linear foot, respectively.

No significant amounts of below-grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of 4 to 6± feet are expected to be necessary to achieve the proposed site grades.

# 4.0 SUBSURFACE EXPLORATION

## 4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of six (6) borings advanced to depths of 20 to 30± feet below the existing site grades. All of the borings were logged during drilling by a member of our staff.

The borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed soil samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. In-situ samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

The approximate locations of the borings are indicated on the Boring Location Plan, included as Plate 2 in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B.

## 4.2 Geotechnical Conditions

### Pavements

Asphaltic concrete pavement was encountered at the ground surface of all boring locations. The pavement sections generally consisted of 3½ to 4½± inches of asphaltic concrete, underlain by 2½ to 7± inches of aggregate base.

### Engineered Fill (Afe)

Engineered fill soils were encountered at Boring Nos. B-1 and B-4 through B-6, extending to depths of 12 to more than 30± feet. Boring Nos. B-1 and B-5 were terminated in the engineered fill soils. These fill materials consist of medium dense to very dense silty sands with varying clay content. Many samples of the fill soils possess variable coloration and variable strength, indicative of their classification as fill.

Capistrano Formation – Oso member (Tco)

Capistrano Formation bedrock was encountered beneath the pavements and/or beneath the fill soils at most of the boring locations, extending to at least the maximum depth explored of 30± feet below the existing site grades. The bedrock generally consists of light gray to dark gray fine-grained poorly consolidated silty sandstone and sandstone. The samples occasionally possess trace amounts of clay, organics and iron oxide staining.

Groundwater

Free water was not encountered during the drilling of any of the borings. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 30± feet at the time of the subsurface exploration.

As a part of our research, we reviewed available groundwater data in order to determine groundwater levels for the site. The primary reference used to determine the groundwater depths in the subject site area is the California State Water Resources Control Board website, GeoTracker, https://geotracker.waterboards.ca.gov. The nearest monitoring well on record is located 1.37± miles west of the site. Water level readings within this monitoring well indicate a groundwater level of 72± feet below the ground surface in September 2017.

## 4.3 Geologic Conditions

Geologic research indicates that the majority of the site is underlain by white to bluish-white, silty, marine, fine- to medium-grained, thick bedded to massive, poorly sorted arkosic sandstone. mapped as late Miocene to early Pliocene Capistrano Formation, Oso Member (Map Symbol Tco). The Holocene to Pleistocene age Slopewash (Map Symbol Qsw) is mapped in the northwestern and southeastern property lines of the site. The bedding within the Capistrano Formation is indicated to trend northwest-southeast with a dip of 14 degrees to the north, on the geologic map. The primary available reference applicable to the subject site is the Geologic Map and Sections of the South Half El Toro Quadrangle, Orange County, California, by Donald Fife, 1974. A portion of this map indicating the location of the subject site is included herein as Plate 3 in Appendix A.

Based on the materials encountered in the exploratory borings, the site is underlain by sandstone, and silty sandstone of the Capistrano Formation.

# 5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. The field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Consolidation

Selected soil samples were tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-4 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plate C-5 in Appendix C of this report. This test is generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Soluble Sulfates

A representative sample of the near-surface soil was submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

| Sample Identification | Soluble Sulfates (%) | Sulfate Classification |
|---|---|---|
| B-1 @ 0 to 5 feet | 0.005 | Not Applicable (S0) |

Corrosivity Testing

One representative sample of the near-surface soils was submitted to a subcontracted corrosion engineering laboratory to identify potentially corrosive characteristics with respect to common construction materials. The corrosivity testing included a determination of the electrical resistivity, pH, and chloride and nitrate concentrations of the soils, as well as other tests. The results of some of these tests are presented below.

| Sample Identification | Saturated Resistivity (ohm-cm) | pH | Chlorides (mg/kg) | Nitrates (mg/kg) |
|---|---|---|---|---|
| B-1 @ 0 to 5 feet | 3,000 | 9.2 | 5.3 | 3.4 |

Expansion Index (EI)

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829. The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50± 1 percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The result of the EI testing is as follows:

| Sample Identification | Expansion Index | Expansive Potential |
|---|---|---|
| B-1 @ 0 to 5 feet | 12 | Very Low |

# 6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations.

The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The recommendations are provided with the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to verify compliance with these recommendations. Maintaining Southern California Geotechnical, Inc., (SCG) as the geotechnical consultant from the beginning to the end of the project will provide continuity of services. The geotechnical engineering firm providing testing and observation services shall assume the responsibility of Geotechnical Engineer of Record.

The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

## 6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site-specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structure should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

### Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Furthermore, SCG did not identify any evidence of faulting during the geotechnical investigations. Therefore, the possibility of significant fault rupture on the site is considered to be low.

The potential for other geologic hazards such as seismically induced settlement, lateral spreading, tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low.

### Seismic Design Parameters

The 2019 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

Based on standards in place at the time of this report, the proposed development is expected to be designed in accordance with the requirements of the 2019 edition of the California Building Code (CBC), which was adopted on January 1, 2020.

The 2019 CBC Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool, a web-based software application available at the website www.seismicmaps.org. This software application calculates seismic design parameters in accordance with several building code reference documents, including ASCE 7-16, upon which the 2019 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake ($MCE_R$) site accelerations at 0.01-degree intervals for each of the code documents. The table below was created using data obtained from the application. The output generated from this program is included as Plate E-1 in Appendix E of this report.

The 2019 CBC requires that a site-specific ground motion study be performed in accordance with Section 11.4.8 of ASCE 7-16 for Site Class D sites with a mapped $S_1$ value greater than 0.2. However, Section 11.4.8 of ASCE 7-16 also indicates an exception to the requirement for a site-specific ground motion hazard analysis for certain structures on Site Class D sites. The commentary for Section 11 of ASCE 7-16 (Page 534 of Section C11 of ASCE 7-16) indicates that "In general, this exception effectively limits the requirements for site-specific hazard analysis to very tall and or flexible structures at Site Class D sites." Based on our understanding of the proposed development, the seismic design parameters presented below were calculated assuming that the exception in Section 11.4.8 applies to the proposed structures at this site. However, the structural engineer should verify that this exception is applicable to the proposed structures. Based on the exception, the spectral response accelerations presented below were calculated using the site coefficients ($F_a$ and $F_v$) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2019 CBC.

2019 CBC SEISMIC DESIGN PARAMETERS

| Parameter | | Value |
|---|---|---|
| Mapped Spectral Acceleration at 0.2 sec Period | $S_S$ | 1.261 |
| Mapped Spectral Acceleration at 1.0 sec Period | $S_1$ | 0.450 |
| Site Class | --- | D |
| Site Modified Spectral Acceleration at 0.2 sec Period | $S_{MS}$ | 1.261 |
| Site Modified Spectral Acceleration at 1.0 sec Period | $S_{M1}$ | 0.833 |
| Design Spectral Acceleration at 0.2 sec Period | $S_{DS}$ | 0.841 |
| Design Spectral Acceleration at 1.0 sec Period | $S_{D1}$ | 0.555 |

It should be noted that the site coefficient $F_v$ and the parameters $S_{M1}$ and $S_{D1}$ were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the 2019 CBC. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2019 CBC using the value of $S_1$

obtained from the Seismic Design Maps Tool, assuming that a site-specific ground motion hazards analysis is not required for the proposed building at this site.

Liquefaction

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and grain size characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean ($d_{50}$) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Clayey (cohesive) soils or soils which possess clay particles (d<0.005mm) in excess of 20 percent (Seed and Idriss, 1982) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The Earthquake Zones of Required Investigation, El Toro Quadrangle map, published by the California Geological Survey (CGS), indicates that the subject site is not located within a designated liquefaction hazard zone. In addition, the subsurface conditions encountered at the subject site are not considered to be conducive to liquefaction. Based on the conditions encountered at the boring locations, and the mapping performed by the CGS, liquefaction is not considered to be a significant design concern for this project. This map also indicates that the site is not located with an Earthquake Induced Landslide Zone.

## 6.2 Geotechnical Design Considerations

General

The subsurface conditions at this site consist of engineered fill soils and moderate to high strength Capistrano formation sandstone at all of the boring locations. The engineered fill soils possess relatively high strengths and favorable consolidation and collapse characteristics. We performed research at the City of Lake Forest in an attempt to obtain reports documenting the placement and compaction of any such fill soils. The City has no such records. We did, however, obtain a copy of a previous geotechnical map for the site, prepared by Petra Geotechnical, which indicates that the site is underlain by engineered fill soils and Capistrano formation bedrock. However, he precise fill depths could not be determined from this plan. All of the data collected by SCG indicates that the existing fill soils represent engineered fill, generally suitable for support of new structures.

As a result of the previous grading, a portion of the proposed warehouse will be underlain by engineered fill soils, whereas the southeastern area of the new building will be underlain by sandstone bedrock. These existing conditions will create a bedrock/fill transition within the proposed building area. It is also expected that the upper 3± feet of soils will be disturbed during demolition of the existing development. Based on these conditions, remedial grading will be necessary within the proposed building area to provide a pad suitable for support of the proposed structure.

The existing pavements are in fair to good condition, and consideration may be given to reusing some of the existing pavements with the new development. However, the existing pavement thicknesses are not adequate to support any significant volume of truck traffic.

Settlement

The recommended remedial grading will remove a portion of the near-surface native bedrock materials and replace these materials as compacted structural fill. Disturbed soils created during demolition of the existing development will also be removed to a stable soil subgrade. The engineered fill soils and bedrock materials that will remain in place below the recommended depth of overexcavation possess relatively high strengths. Therefore, following completion of the recommended grading, post-construction settlements are expected to be within tolerable limits.

Expansion

The near-surface soils consist of silty sandstone with no appreciable clay content. The results of expansion index testing indicate that these materials are very low expansive (EI = 12). Therefore, no design considerations related to expansive soils are considered warranted for this site.

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected sample of the on-site soils contains a sulfate concentration that corresponds to Class S0 with respect to the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building areas.

Corrosion Potential

The results of laboratory testing indicate that the tested sample of the on-site soils possesses a saturated resistivity value of 3,000 ohm-cm, and a pH value of 9.2. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity and pH are two of the five factors that enter into the evaluation procedure. Redox potential, relative soil moisture content and sulfides are also included. Although sulfide testing was not part of the scope of services for this project, we have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH and moisture content. Based on these factors, and utilizing the DIPRA procedure, the on-site soils are considered to be moderately corrosive to ductile iron pipe. Therefore, polyethylene protection may be required for cast iron or ductile iron pipes.

A relatively low concentration (5.3 mg/kg) of chlorides was detected in the sample submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the lack of any significant chlorides in the tested sample, the site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for reinforced concrete for protection against chloride exposure is not considered warranted.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested sample possesses a nitrate concentration of 3.4 mg/kg. Based on this test result, the on-site soils are not considered to be corrosive to copper pipe. Since SCG does not practice in the area of corrosion engineering, the client may wish to contact a corrosion engineer to provide a more thorough evaluation.

It should be noted that SCG does not practice in the field of corrosion engineering. Therefore, the client may wish to contact a corrosion engineer to provide a more thorough evaluation.

Shrinkage/Subsidence

Removal and recompaction of the existing fill soils is estimated to result in an average shrinkage of 3 to 10 percent. Bedrock materials are expected to result in less than 5% shrinkage or bulking when removed and replaced as compacted fill. These shrinkage/bulking estimates are based on the assumption that the onsite soils will be compacted to about 92 percent of the ASTM D-1557 maximum dry density. It should be noted that these estimates are based on the results of dry density testing performed on small-diameter samples of the existing soils taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1 feet. This estimate may be used for grading in areas that are underlain by existing engineered fill soils.

These estimates are based on previous experience and the subsurface conditions encountered at the boring locations. The actual amount of subsidence is expected to be variable and will be dependent on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

Detailed grading and foundation plans were not available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

## 6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations, and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping and Demolition

Demolition of the existing structure should include any improvements that will not remain in place for use with the new development, including foundations, floor slabs, and utilities. Any pavements that will be reused with the new development should be protected from damage by construction traffic. Debris resultant from demolition should be disposed of off-site. All applicable federal, state and local specifications and regulations should be followed in demolition, abandonment, and disposal of the resulting debris. Concrete and asphalt debris may be crushed to a maximum 2-inch particle size, well-mixed with the on-site soils, and incorporated into new structural fills.

Initial site stripping should include removal of any vegetation, as well as any underlying topsoil or other organic materials from landscaped areas. Based on conditions observed at the time of the subsurface exploration, stripping of grass, shrubs, and trees will be required. Root masses associated with the trees should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils. Any organic materials should be removed and disposed of off-site, or in non-structural areas of the property. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building area in order to remove the soils disturbed during demolition and to mitigate the bedrock/fill transitions that would otherwise exist. Based on conditions encountered at the boring locations, the existing soils/bedrock within the proposed building area are recommended to be overexcavated to a depth of at least 5 feet below existing grades and to a depth of at least 3 feet below proposed building pad subgrade elevations, whichever is greater. The depth of the overexcavation should also extend to a depth sufficient to remove all undocumented fill soils and soils disturbed during demolition. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.

Additional overexcavation should be performed in the southeastern area of the proposed building pad, in the area of Boring Nos. B-2 and B-3, to soften the bedrock/fill transition that will exist in this area of the site. This area of the pad should be overexcavated to a depth of 5 feet below foundation bearing grade. The extent of the shallow bedrock in this area should be confirmed during grading.

The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters, and to an extent equal to the depth of fill placed below the foundation bearing grade, whichever is greater. If the proposed structure incorporates any exterior columns (such as for a canopy or overhang) the overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the overexcavation areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, or low-density native soils are encountered at the base of the overexcavation.

After a suitable overexcavation subgrade has been achieved, the exposed soils/bedrock should be scarified to a depth of at least 12 inches and moisture treated to 0 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The building pad area may then be raised to grade with previously excavated soils or imported structural fill.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls and site walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad. Retaining wall or site wall foundations may also be supported within sandstone bedrock materials with no further overexcavation. Any disturbed or low strength fill soils within any of these foundation areas should be removed in their entirety. The overexcavation areas should extend at least 3 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. Any erection pads for tilt-up concrete walls are considered to be part of the foundation system. Therefore, these overexcavation recommendations are applicable to erection pads. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning to within 0 to 4 percent above the optimum moisture content, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral recommended remedial grading cannot be completed for the proposed retaining walls and site walls located along property lines, the foundations for those walls should be designed using a reduced allowable bearing pressure. Furthermore, the contractor should take necessary precautions to protect the adjacent improvements during rough grading. Specialized grading techniques, such as A-B-C slot cuts, will likely be required during remedial grading. The geotechnical engineer of record should be contacted if additional recommendations, such as shoring design recommendations, are required during grading.

Treatment of Existing Soils: Flatwork, Parking and Drive Areas

Based on economic considerations, overexcavation of the existing near-surface existing soils in the new flatwork, parking and drive areas is not considered warranted, with the exception of areas where lower strength or unstable soils are identified by the geotechnical engineer during grading. Subgrade preparation in the new flatwork, parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations.

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above the optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 0 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2019 CBC and the grading code of the city of Lake Forest.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of very low expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Lake Forest. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v (horizontal to vertical) plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

Any soils used to backfill voids around subsurface utility structures, such as manholes or vaults, should be placed as compacted structural fill. If it is not practical to place compacted fill in these areas, then such void spaces may be backfilled with lean concrete slurry. Uncompacted pea gravel or sand is not recommended for backfilling these voids since these materials have a potential to settle and thereby cause distress of pavements placed around these subterranean structures.

## 6.4 Construction Considerations

### Excavation Considerations

The near-surface soils generally consist of sands and silty sands. These materials may be subject to moderate caving within shallow excavations. Where caving does occur, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 2h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

### Groundwater

The static groundwater table is considered to have existed at a depth in excess of 30± feet at the time of the subsurface exploration. Therefore, groundwater is not expected to impact grading or foundation construction activities.

## 6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils used to replace a portion of the existing fill soils, and the existing sandstone bedrock. These new structural fill soils are expected to extend to depths of at least 3 to 5 feet below proposed foundation bearing grade, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, the proposed structure may be supported on conventional shallow foundations.

### Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 3,000 lbs/ft$^2$.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Four (4) No. 5 rebars (2 top and 2 bottom).
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on geotechnical considerations; additional reinforcement may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill or suitable native alluvium (where reduced bearing pressures are utilized), with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 0 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slab and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 $lbs/ft^3$
- Friction Coefficient: 0.30

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill soils. The maximum allowable passive pressure is 3,000 $lbs/ft^2$.

## 6.6 Floor Slab Design and Construction

Subgrades which will support the new floor slabs should be prepared in accordance with the recommendations contained in the *Site Grading Recommendations* section of this report. Based on the anticipated grading which will occur at this site, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill (or densified existing soils), extending to a depth of at least 3 feet below finished pad grades. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.
- Modulus of Subgrade Reaction: k = 150 psi/in.
- Minimum slab reinforcement: Reinforcement is not considered necessary from a geotechnical standpoint. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.
- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab where such moisture sensitive floor coverings are anticipated. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.
- Moisture condition the floor slab subgrade soils to 0 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

## 6.7 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for sidewalks, patios, and other concrete flatwork, should be prepared in accordance with the recommendations contained in the

*Grading Recommendations* section of this report. Assuming that the flatwork is supported on the existing soils, exterior slabs-on-grade may be designed as follows:

- Minimum slab thickness: 4½ inches.
- Minimum slab reinforcement: No. 3 bars at 18 inches on center, in both directions.
- The flatwork at building entry areas should be structurally connected to the perimeter foundation that is recommended to span across the door opening. This recommendation is designed to reduce the potential for differential movement at this joint.
- Moisture condition the slab subgrade soils to within 0 to 4 percent of optimum moisture content, to a depth of at least 12 inches. Adequate moisture conditioning should be verified by the geotechnical engineer 24 hours prior to concrete placement.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.
- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 6 feet on center for sidewalks. Control joints are intended to direct cracking. Minor cracking of exterior concrete slabs on grade should be expected.

Expansion or felt joints should be used at the interface of exterior slabs on grade and any fixed structures to permit relative movement.

## 6.8 Retaining Wall Design and Construction

Although not indicated on the site plans, some small (less than 6 feet in height) retaining walls may be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

### Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. The following parameters assume that only the on-site soils will be utilized for retaining wall backfill. The near-surface soils generally consist of sands and silty sands. Based on their classification, these materials are expected to possess a friction angle of at least 30 degrees when compacted to at least 90 percent of the ASTM D-1557 maximum dry density.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

RETAINING WALL DESIGN PARAMETERS

| Design Parameter | | Soil Type: On-site Sands and Silty Sands |
|---|---|---|
| Internal Friction Angle ($\phi$) | | 30° |
| Unit Weight | | 128 lbs/ft$^3$ |
| Equivalent Fluid Pressure: | Active Condition (level backfill) | 43 lbs/ft$^3$ |
| | Active Condition (2h:1v backfill) | 69 lbs/ft$^3$ |
| | At-Rest Condition (level backfill) | 64 lbs/ft$^3$ |

The walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft$^3$. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Seismic Lateral Earth Pressures

In accordance with the 2019 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be underlain by at least 3 feet of newly placed structural fill, or undisturbed Capistrano formation bedrock.  Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site soils may be used to backfill the retaining walls. However, all backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. Some sorting and/or crushing operations may be required. The retaining wall backfill materials should be well graded.

It is recommended that a minimum 1-foot thick layer of free-draining granular material (less than 5 percent passing the No. 200 sieve) be placed against the face of the retaining walls. This material should extend from the top of the retaining wall footing to within 1 foot of the ground surface on the back side of the retaining wall. This material should be approved by the geotechnical engineer. In lieu of the 1-foot thick layer of free-draining material, a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls, may be used. If the layer of free-draining material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The layer of free draining granular material should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557-91). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

## 6.9 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the *Site Grading Recommendations* section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Existing Pavements

The existing pavements are generally in good condition and are suitable for reuse with the new development. However, these pavements were designed for the traffic associated with the existing office building and are consequently relatively thin sections. The pavements consist of 3½ to 4½ inches of AC over 2½ to 7 inches of AB. These pavements are only considered suitable for reuse in auto and light truck traffic areas. If these pavements are subjected to significant heavy truck traffic, they will likely experience a short service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of sands and silty sands. These soils are generally considered to possess good to excellent pavement support characteristics, with R-values in the range of 40 to 60. The subsequent pavement design is therefore based upon an assumed R-value of 40. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading to verify that the pavement design recommendations presented herein are valid.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20-year design life, assuming six operational traffic days per week.

| Traffic Index | No. of Heavy Trucks per Day |
|---|---|
| 4.0 | 0 |
| 5.0 | 1 |
| 6.0 | 3 |
| 7.0 | 11 |
| 8.0 | 35 |

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

| ASPHALT PAVEMENTS (R = 40) | | | | | |
|---|---|---|---|---|---|
| Materials | Thickness (inches) | | | | |
| | Parking Stalls (TI = 4.0) | Auto Drive Lanes (TI = 5.0) | Truck Traffic | | |
| | | | (TI = 6.0) | (TI = 7.0) | (TI = 8.0) |
| Asphalt Concrete | 3 | 3 | 3½ | 4 | 5 |
| Aggregate Base | 3 | 4 | 6 | 7 | 8 |
| Compacted Subgrade (90% minimum compaction) | 12 | 12 | 12 | 12 | 12 |

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the batch plant-reported maximum density. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

| PORTLAND CEMENT CONCRETE PAVEMENTS (R = 40) | | | | |
|---|---|---|---|---|
| Materials | Thickness (inches) | | | |
| | Automobile Parking and Drive Areas (TI = 5.0) | Truck Traffic | | |
| | | (TI =6.0) | (TI =7.0) | (TI =8.0) |
| PCC | 5 | 5 | 5½ | 6½ |
| Compacted Subgrade (95% minimum compaction) | 12 | 12 | 12 | 12 |

The concrete should have a 28-day compressive strength of at least 3,000 psi. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness. Any reinforcement within the PCC pavements should be determined by the project structural engineer.

# 7.0 GENERAL COMMENTS

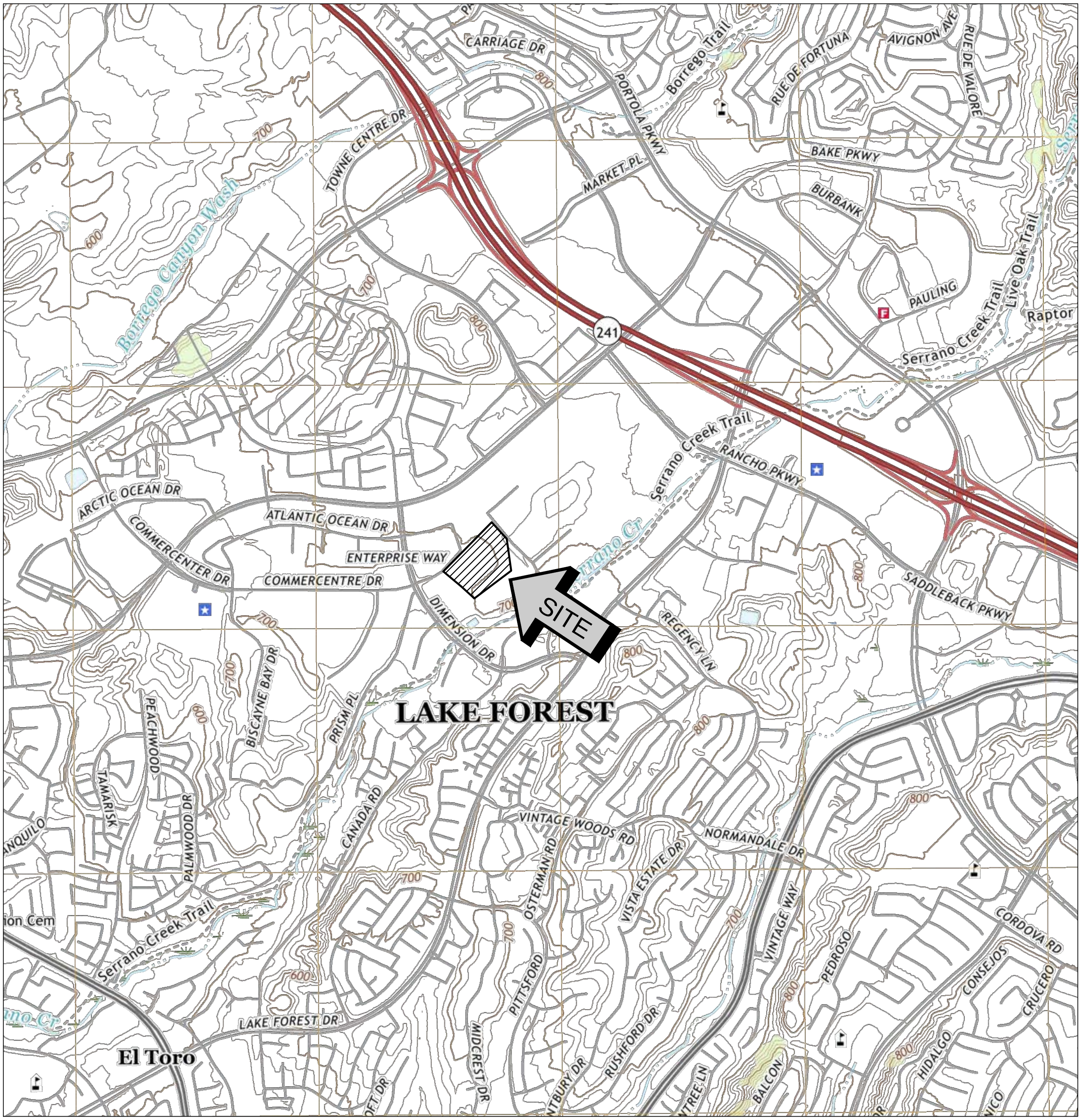
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between boring locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

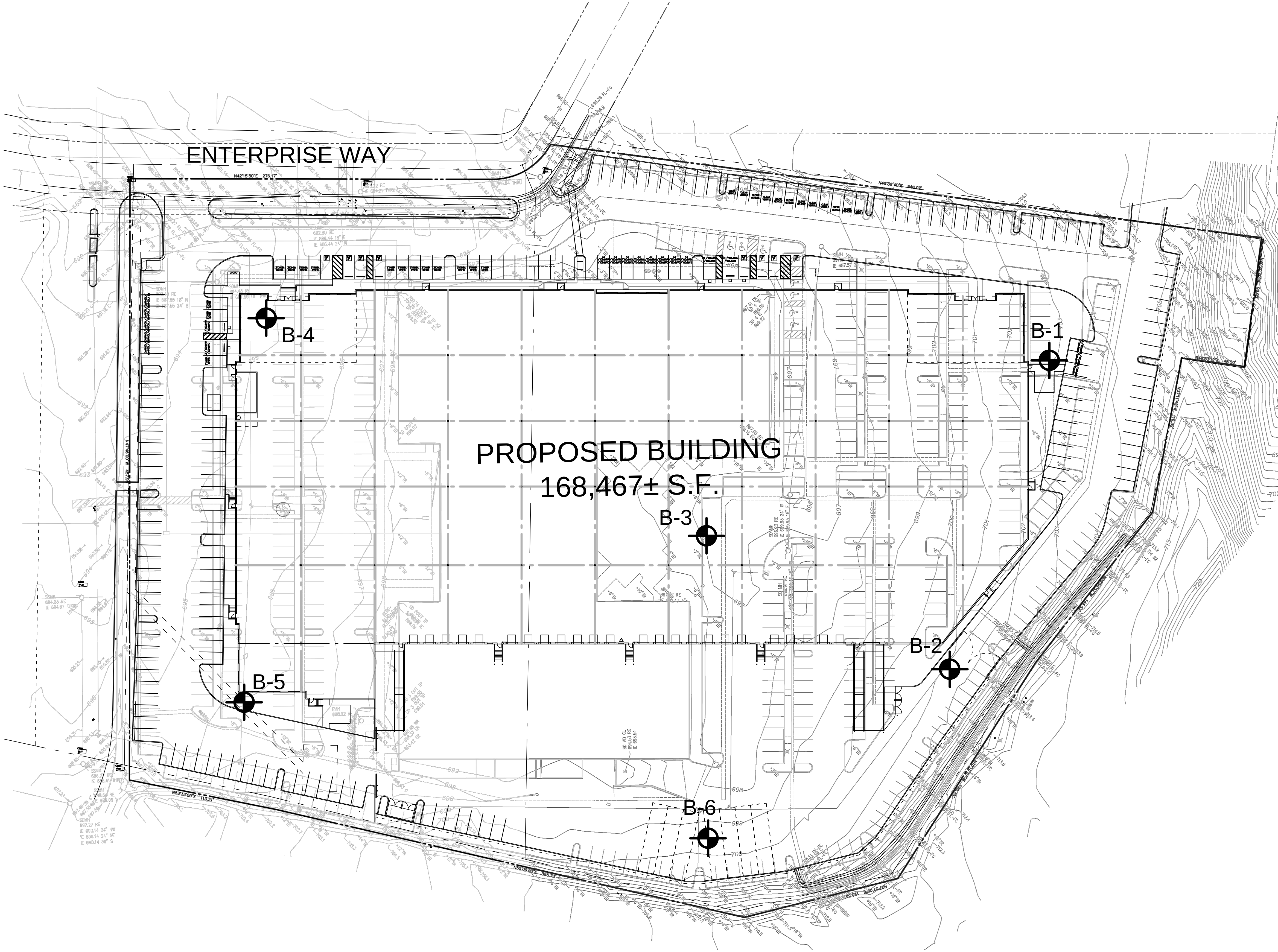
The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

# APPENDIX A




SOURCE: USGS TOPOGRAPHIC MAP OF LAKE FOREST QUADRANGLE, ORANGE COUNTY, CALIFORNIA, 2018

| SITE LOCATION MAP | |
|---|---|
| PROPOSED WAREHOUSE | |
| LAKE FOREST, CALIFORNIA | |
| SCALE: 1" = 2000' | SOUTHERN CALIFORNIA GEOTECHNICAL |
| DRAWN: MD<br>CHKD: RGT | |
| SCG PROJECT<br>21G135-1 | |
| **PLATE 1** | |




GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

NOTE: BASE MAP PREPARED BY KIER + WRIGHT

| BORING LOCATION PLAN | |
|---|---|
| PROPOSED WAREHOUSE | |
| LAKE FOREST, CALIFORNIA | |
| SCALE: 1" = 80' | |
| DRAWN: JAH<br>CHKD: RGT | |
| SCG PROJECT<br>21G135-1R | |
| PLATE 2 | |


LEGEND

**Qsw**

*SLOPEWASH. (Holocene to Pleistocene(?))*

*Lithology*: Varies with parent materials. In general consists of light olive-gray to olive-gray, silty to clayey, organicly rich soil which is plastic when wet and hard when dry. Consists of detrital materials resulting from debris flows or creep aided by rill wash, ground water, and unchanneled surficial drainage. Accumulates near the base of slopes and at the mouths of small canyons. Includes colluvium and minor residu soils. It is commonly transitional with adjacent soils or alluvial deposits. Mapped where estimated to exceed three feet in thickness.

**Tco**

*CAPISTRANO FORMATION, OSO MEMBER. (late Miocene to early Pliocene)*

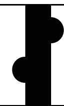
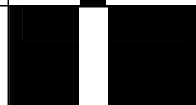
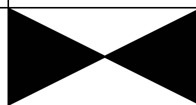
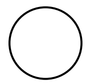
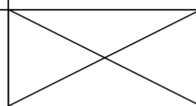
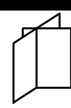
*Lithology*: Marine, white to bluish-white, silty, fine-to medium-grained, thick bedded to massive, poorly sorted arkosic sandstone. Commonly cross-bedded. Locally contains calcareous concretions as much as two feet in diameter. Concretionary zones commonly are found in association with marine vertebrate fossils, but locally friable sand lenses are encountered. Bentonite beds 1 to 2 inches thick, were observed in an anticline on the east flank of Aliso Creek.

SOURCE: "GEOLOGY MAP OF THE SOUTH HALF OF THE EL TORO QUADRANGLE, ORANGE COUNTY, CALIFORNIA" BY DONALD L. FIFE, 1974

| **GEOLOGIC MAP** | |
|---|---|
| PROPOSED WAREHOUSE | |
| LAKE FOREST, CALIFORNIA | |
| SCALE: 1" = 2000' | SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL |
| DRAWN: JAH<br>CHKD: GKM | |
| SCG PROJECT<br>21G135-1 | |
| **PLATE 3** | |

# APPENDIX B

# BORING LOG LEGEND

| SAMPLE TYPE | GRAPHICAL SYMBOL | SAMPLE DESCRIPTION |
|---|---|---|
| AUGER |  | SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED) |
| CORE |  | ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK. |
| GRAB |  | SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED) |
| CS |  | CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED) |
| NSR |  | NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL. |
| SPT |  | STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED) |
| SH |  | SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED) |
| VANE |  | VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED. |

## COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

BLOW COUNT: Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3” indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.

POCKET PEN.: Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.

GRAPHIC LOG: Graphic Soil Symbol as depicted on the following page.

DRY DENSITY: Dry density of an undisturbed or relatively undisturbed sample in $lbs/ft^3$.

MOISTURE CONTENT: Moisture content of a soil sample, expressed as a percentage of the dry weight.

LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

PLASTIC LIMIT: The moisture content above which a soil behaves as a plastic.

PASSING #200 SIEVE: The percentage of the sample finer than the #200 standard sieve.

UNCONFINED SHEAR: The shear strength of a cohesive soil sample, as measured in the unconfined state.

# SOIL CLASSIFICATION CHART

| MAJOR DIVISIONS | | | SYMBOLS GRAPH | SYMBOLS LETTER | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|---|
| COARSE GRAINED SOILS<br>MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE | GRAVEL AND GRAVELLY SOILS<br>MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE | CLEAN GRAVELS<br>(LITTLE OR NO FINES) | | **GW** | WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | | | | **GP** | POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | | GRAVELS WITH FINES<br>(APPRECIABLE AMOUNT OF FINES) | | **GM** | SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES |
| | | | | **GC** | CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES |
| | SAND AND SANDY SOILS<br>MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE | CLEAN SANDS<br>(LITTLE OR NO FINES) | | **SW** | WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES |
| | | | | **SP** | POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES |
| | | SANDS WITH FINES<br>(APPRECIABLE AMOUNT OF FINES) | | **SM** | SILTY SANDS, SAND - SILT MIXTURES |
| | | | | **SC** | CLAYEY SANDS, SAND - CLAY MIXTURES |
| FINE GRAINED SOILS<br>MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE | SILTS AND CLAYS | LIQUID LIMIT LESS THAN 50 | | **ML** | INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY |
| | | | | **CL** | INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS |
| | | | | **OL** | ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY |
| | SILTS AND CLAYS | LIQUID LIMIT GREATER THAN 50 | | **MH** | INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS |
| | | | | **CH** | INORGANIC CLAYS OF HIGH PLASTICITY |
| | | | | **OH** | ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS |
| HIGHLY ORGANIC SOILS | | | | **PT** | PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS |

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-1

JOB NO.: 21G135-1
PROJECT: Proposed Warehouse
LOCATION: Lake Forest, California

DRILLING DATE: 3/12/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 28 feet
READING TAKEN: At Completion

SURFACE ELEVATION: 702.5 feet MSL

| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 3½± inches Asphaltic Concrete; 3½± inches Aggregate Base | | | | | | | |
| | X | 21 | | | ENGINEERED FILL (Afe): Light Gray to Gray Brown Silty fine Sand, little Iron oxide staining, medium dense to dense-moist to very moist | | 15 | | | | | EI = 12 @ 0 to 5 feet |
| | X | 29 | | | | | 15 | | | | | |
| 5 | | | | | | | | | | | | |
| | X | 25 | | | | | 12 | | | | | |
| | X | 30 | | | | | 12 | | | | | |
| 10 | | | | | | | | | | | | |
| | X | 29 | | | | | 14 | | | | | |
| 15 | | | | | | | | | | | | |
| | | | | | ENGINEERED FILL (Afe): Dark Gray Silty fine Sand, little Clay, medium dense-moist to very moist | | | | | | | |
| | X | 23 | | | | | 15 | | | | | |
| 20 | | | | | | | | | | | | |
| | X | 18 | | | | | 15 | | | | | |
| 25 | | | | | | | | | | | | |
| | | | | | @ 28 to 30 feet, Dark Gray Brown, some Clay, dense | | | | | | | |
| | X | 31 | | | | | 14 | | | | | |
| 30 | | | | | | | | | | | | |
| | | | | | Boring Terminated at 30' | | | | | | | |

TBL 21G135-1.GPJ SOCALGEO.GDT 4/8/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-2

JOB NO.: 21G135-1
PROJECT: Proposed Warehouse
LOCATION: Lake Forest, California

DRILLING DATE: 3/12/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 23 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION<br>SURFACE ELEVATION: 701.5 feet MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | 4½± inches Asphaltic Concrete; 7± inches Aggregate Base | | | | | | | |
| | | 78/10" | | | CAPISTRANO FORMATION (Tco): Light Gray fine-grained Sandstone, very dense-damp to moist | 108 | 10 | | | | | |
| | | 93/9" | | | @ 3 feet, little medium Sand | 111 | 7 | | | | | |
| 5 | | 50/5" | | | | 101 | 8 | | | | | |
| | | 50/5" | | | | 98 | 8 | | | | | |
| 10 | | 50/5" | | | | 99 | 6 | | | | | |
| 15 | | 88/9" | | | | | 10 | | | | | |
| 20 | | 76/10" | | | | | 13 | | | | | |
| 25 | | 90/11" | | | @ 23½ feet, little Iron oxide staining | | 7 | | | | | |
| | | | | | Boring Terminated at 25' | | | | | | | |

TBL 21G135-1.GPJ SOCALGEO.GDT 4/8/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-3

| | | |
|---|---|---|
| JOB NO.: 21G135-1 | DRILLING DATE: 3/12/21 | WATER DEPTH: Dry |
| PROJECT: Proposed Warehouse | DRILLING METHOD: Hollow Stem Auger | CAVE DEPTH: 17 feet |
| LOCATION: Lake Forest, California | LOGGED BY: Jamie Hayward | READING TAKEN: At Completion |

| FIELD RESULTS | | | | | | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION<br>SURFACE ELEVATION: 697 feet MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | 3½± inches Asphaltic Concrete; 3± inches Aggregate Base | | | | | | | |
| | | 77/10" | | | CAPISTRANO FORMATION (Tco): Light Gray fine-grained Sandstone, very dense-damp to moist | 110 | 4 | | | | | |
| | | 50/5" | | | | 98 | 6 | | | | | |
| 5 | | 50/5.5" | | | | 104 | 5 | | | | | |
| | | 50/5" | | | | 100 | 7 | | | | | |
| | | 50/5" | | | | 96 | 7 | | | | | |
| 10 | | | | | | | | | | | | |
| | | 83/9" | | | | | 8 | | | | | |
| 15 | | | | | | | | | | | | |
| | | 91/10" | | | @ 18½ feet, Gray Brown, trace Silt | | 10 | | | | | |
| 20 | | | | | | | | | | | | |
| | | | | | Boring Terminated at 20' | | | | | | | |

TBL 21G135-1.GPJ SOCALGEO.GDT 4/8/21

# BORING NO. B-4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 21G135-1
PROJECT: Proposed Warehouse
LOCATION: Lake Forest, California

DRILLING DATE: 3/12/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 23 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION<br>SURFACE ELEVATION: 695 feet MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | 4± inches Asphaltic Concrete; 2½± inches Aggregate Base | | | | | | | |
| | | 47 | | | ENGINEERED FILL (Afe): Gray Brown Silty fine Sand, little Iron oxide staining, medium dense to dense-moist to very moist | 113 | 14 | | | | | |
| | | 36 | | | | 109 | 12 | | | | | |
| 5 | | 43 | | | | 110 | 11 | | | | | |
| | | 40 | | | | 109 | 14 | | | | | |
| 10 | | 77/11" | | | @ 9 feet, trace Clay, very dense | 118 | 13 | | | | | |
| | | | | | ENGINEERED FILL (Afe): Gray to Gray Brown Silty fine Sand, trace Clay, varied coloration, dense-very moist | | | | | | | |
| 15 | | 33 | | | | | 17 | | | | | |
| 20 | | 33 | | | | | 16<br>13 | | | | | |
| | | | | | CAPISTRANO FORMATION (Tco): Light Gray Brown fine-grained Sandstone, very dense-moist | | | | | | | |
| 25 | | 75/11" | | | | | 10 | | | | | |

Boring Terminated at 25'

TBL 21G135-1.GPJ SOCALGEO.GDT 4/8/21

**TEST BORING LOG**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-5

JOB NO.: 21G135-1
PROJECT: Proposed Warehouse
LOCATION: Lake Forest, California

DRILLING DATE: 3/12/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 26 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION<br>SURFACE ELEVATION: 696 feet MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | 3½± inches Asphaltic Concrete; 3± inches Aggregate Base | | | | | | | |
| | X | 37 | | | ENGINEERED FILL (Afe): Light Gray Brown to Gray Brown Silty fine Sand, trace Clay, little Iron oxide staining, varied coloration, medium dense to dense-moist to very moist | | 13 | | | | | |
| | X | 20 | | | | | 12 | | | | | |
| 5 | | | | | | | | | | | | |
| | X | 35 | | | | | 14 | | | | | |
| | X | 26 | | | ENGINEERED FILL (Afe): Light Gray Brown Silty fine Sand, little Clay, little Iron oxide staining, varied coloration, medium dense to very dense-moist to very moist | | 12 | | | | | |
| 10 | | | | | | | | | | | | |
| | X | 33 | | | | | 13 | | | | | |
| 15 | | | | | | | | | | | | |
| | X | 61/9" | | | | | 15 | | | | | |
| 20 | | | | | | | | | | | | |
| | X | 28 | | | | | 14 | | | | | |
| 25 | | | | | | | | | | | | |
| | | | | | ENGINEERED FILL (Afe): Dark Brown to Gray Brown Clayey fine Sand, varied coloration, dense-moist | | | | | | | |
| | X | 36 | | | | | 14 | | | | | |
| 30 | | | | | | | | | | | | |
| | | | | | Boring Terminated at 30' | | | | | | | |

TBL 21G135-1.GPJ SOCALGEO.GDT 4/8/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-6

JOB NO.: 21G135-1
PROJECT: Proposed Warehouse
LOCATION: Lake Forest, California

DRILLING DATE: 3/12/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 17 feet
READING TAKEN: At Completion

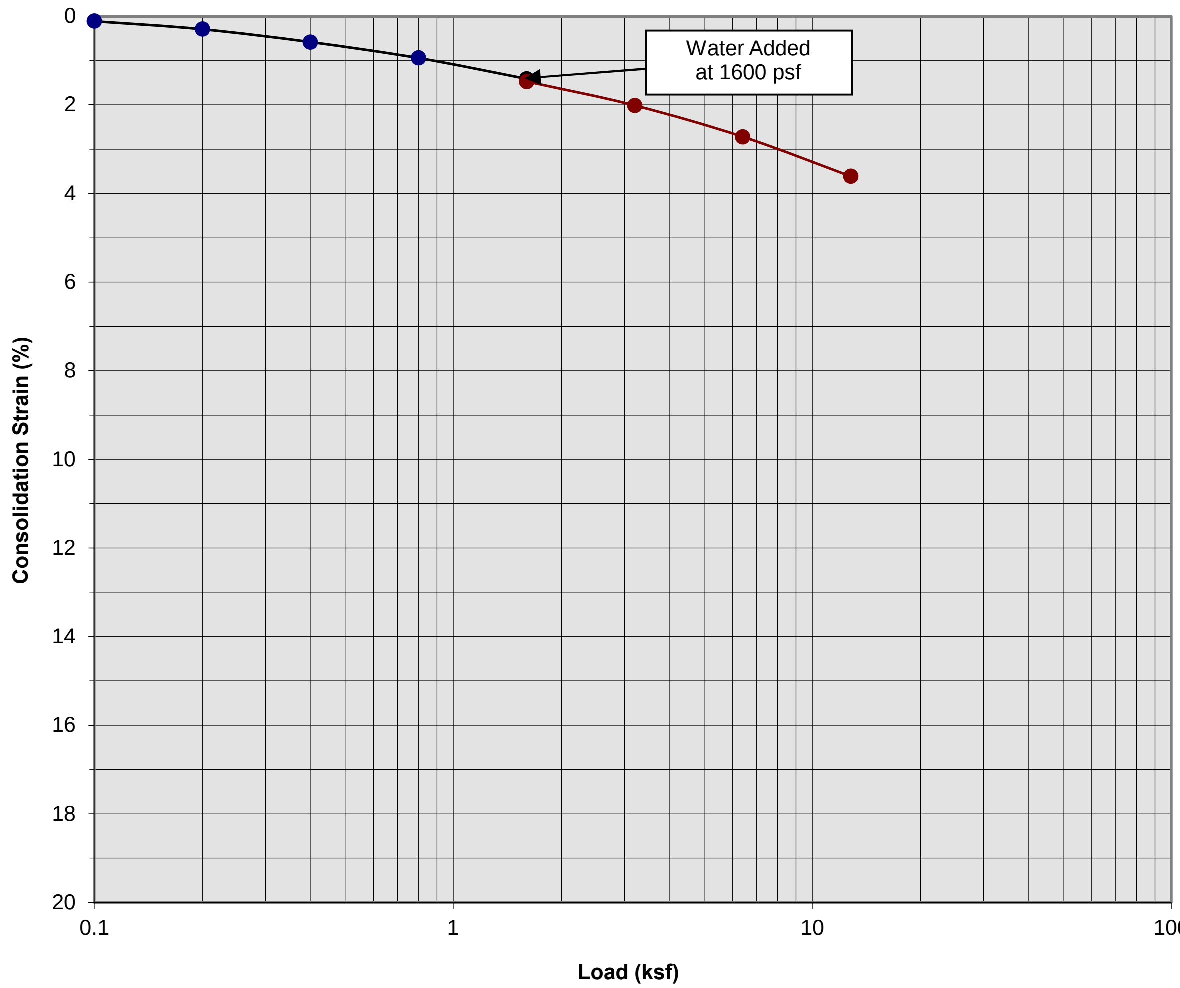
SURFACE ELEVATION: 699.5 feet MSL

| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 4½± inches Asphaltic Concrete; 4½± inches Aggregate Base | | | | | | | |
| | | | | | <u>ENGINEERED FILL (Afe):</u> Light Gray Brown Silty fine Sand, trace medium Sand, trace Clay, medium dense to very dense-moist to very moist | | 12 | | | | | Hand Augered 0 to 5 feet due to Existing Utility |
| 5 | | | | | | | 10 | | | | | |
| | | 50 | | | @ 6 to 8½ feet, trace to little Iron oxide staining | | 14 | | | | | |
| | | 36 | | | | | 13 | | | | | |
| 10 | | | | | | | | | | | | |
| | | | | | <u>CAPISTRANO FORMATION (Tco):</u> Gray Brown fine-grained Silty Sandstone, little Iron oxide staining, poorly consolidated, dense to very dense-moist | | | | | | | |
| | | 39 | | | | | 11 | | | | | |
| 15 | | | | | | | | | | | | |
| | | 56 | | | | | 12 | | | | | |
| 20 | | | | | | | | | | | | |

Boring Terminated at 20'

TBL 21G135-1.GPJ SOCALGEO.GDT 4/8/21

# APPENDIX C




Classification: FILL: Gray Brown Silty fine Sand, trace Clay

| | | | |
|---|---|---|---|
| Boring Number: | B-4 | Initial Moisture Content (%) | 12 |
| Sample Number: | --- | Final Moisture Content (%) | 14 |
| Depth (ft) | 3 to 4 | Initial Dry Density (pcf) | 109.6 |
| Specimen Diameter (in) | 2.4 | Final Dry Density (pcf) | 114.2 |
| Specimen Thickness (in) | 1.0 | Percent Collapse (%) | 0.06 |

Proposed Warehouse
Lake Forest, California
Project No. 21G135-1
**PLATE C- 1**

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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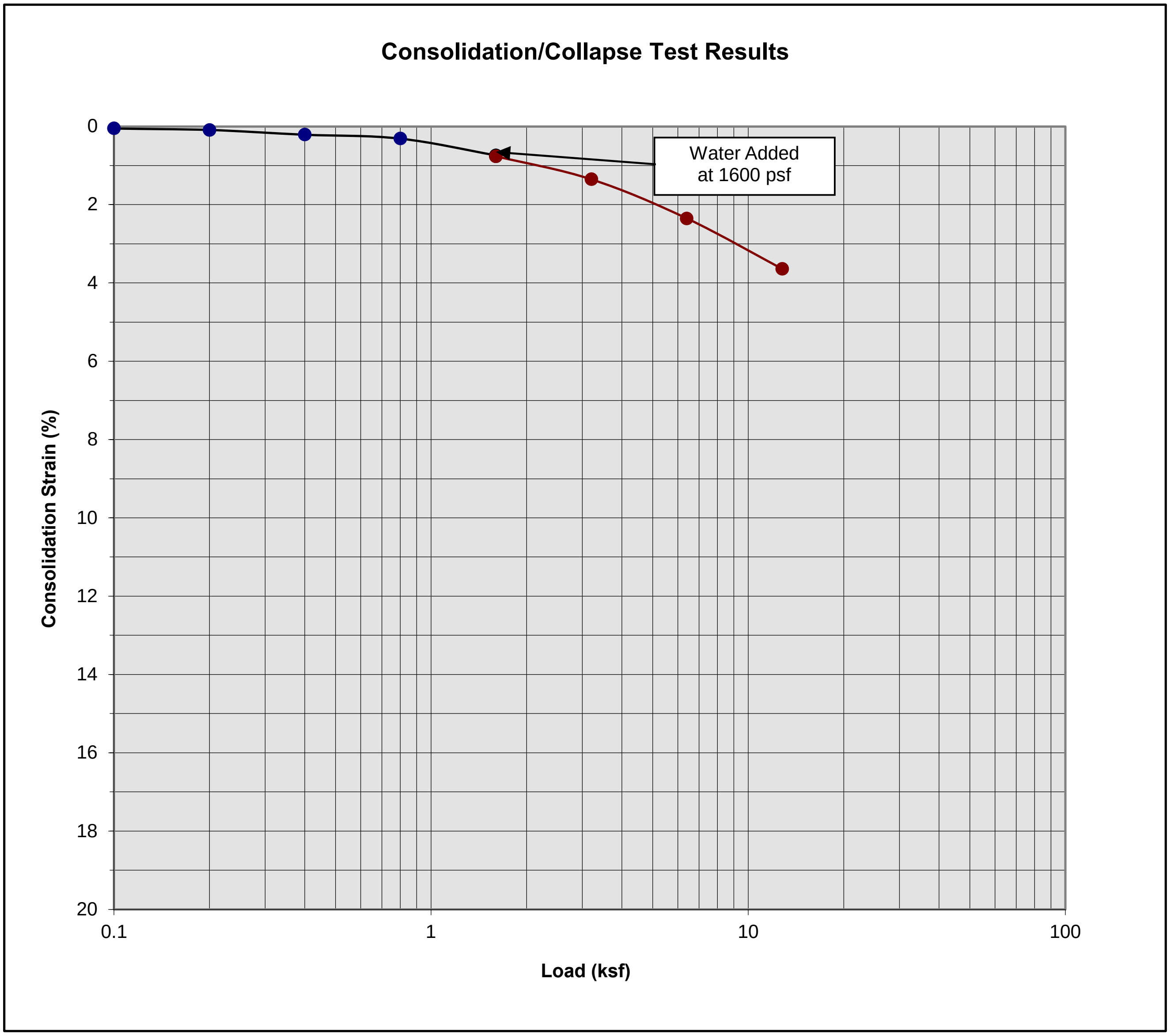


Classification: FILL: Gray Brown Silty fine Sand, trace Clay

| | | | |
|---|---|---|---|
| Boring Number: | B-4 | Initial Moisture Content (%) | 11 |
| Sample Number: | --- | Final Moisture Content (%) | 16 |
| Depth (ft) | 5 to 6 | Initial Dry Density (pcf) | 109.9 |
| Specimen Diameter (in) | 2.4 | Final Dry Density (pcf) | 114.7 |
| Specimen Thickness (in) | 1.0 | Percent Collapse (%) | 0.02 |

Proposed Warehouse
Lake Forest, California
Project No. 21G135-1
**PLATE C- 2**

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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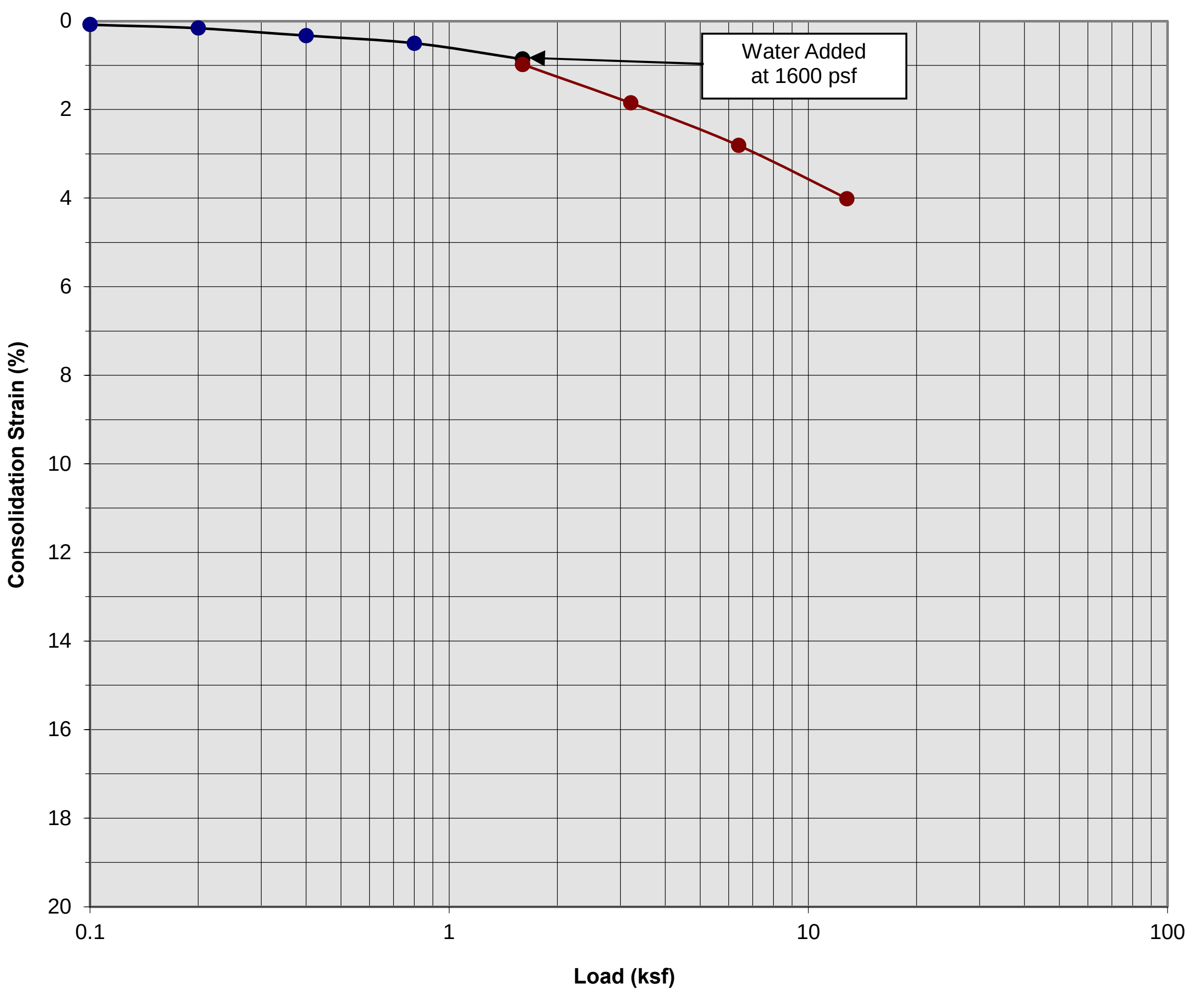
## Consolidation/Collapse Test Results




Classification: FILL: Gray Brown Silty fine Sand, trace Clay

| | | | |
|---|---|---|---|
| Boring Number: | B-4 | Initial Moisture Content (%) | 13 |
| Sample Number: | --- | Final Moisture Content (%) | 17 |
| Depth (ft) | 7 to 8 | Initial Dry Density (pcf) | 108.8 |
| Specimen Diameter (in) | 2.4 | Final Dry Density (pcf) | 113.2 |
| Specimen Thickness (in) | 1.0 | Percent Collapse (%) | 0.12 |

Proposed Warehouse
Lake Forest, California
Project No. 21G135-1
**PLATE C- 3**

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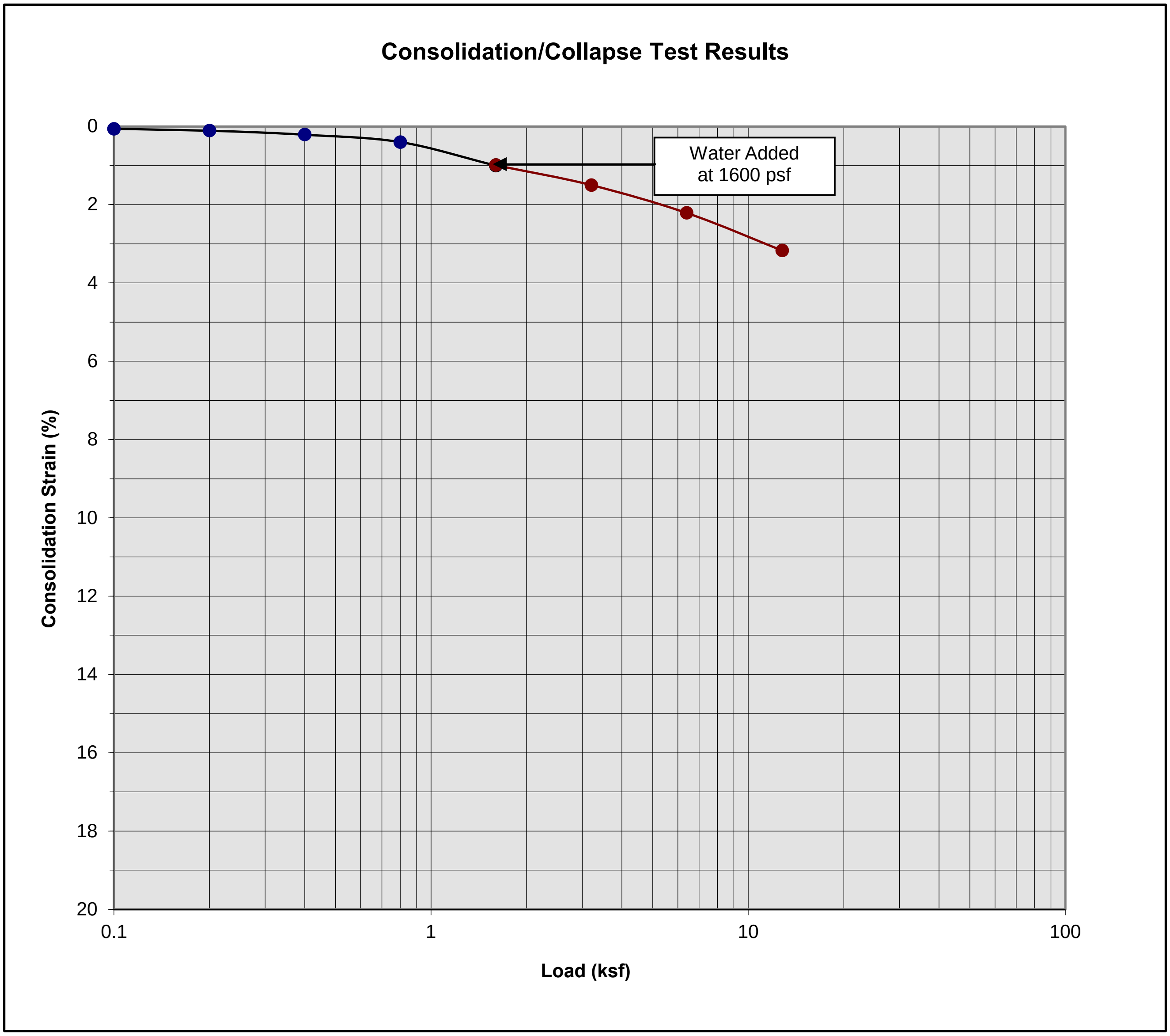


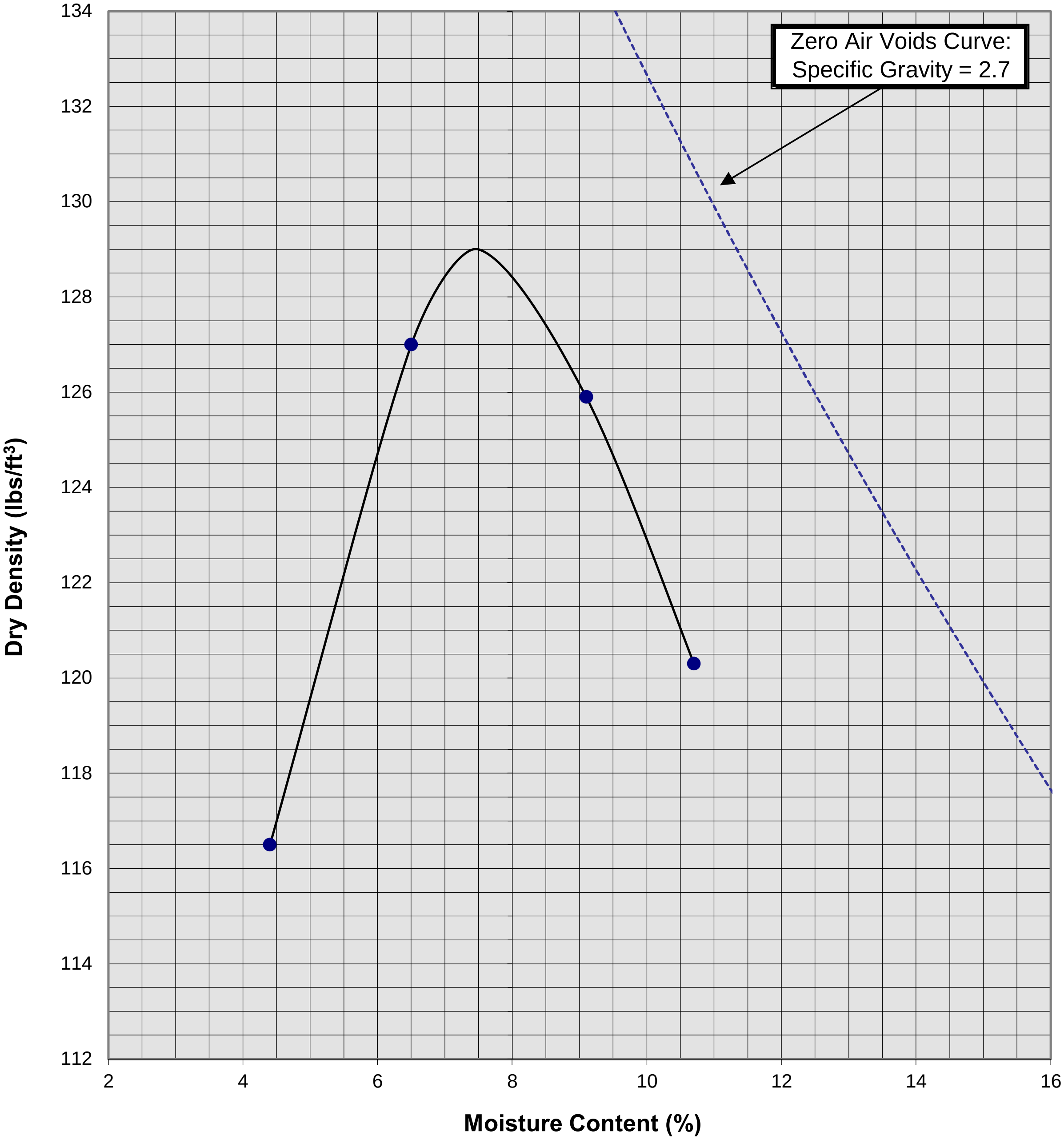

Classification: FILL: Gray Brown Silty fine Sand, trace Clay

| | | | |
|---|---|---|---|
| Boring Number: | B-4 | Initial Moisture Content (%) | 13 |
| Sample Number: | --- | Final Moisture Content (%) | 16 |
| Depth (ft) | 9 to 10 | Initial Dry Density (pcf) | 117.9 |
| Specimen Diameter (in) | 2.4 | Final Dry Density (pcf) | 122.2 |
| Specimen Thickness (in) | 1.0 | Percent Collapse (%) | -0.01 |

Proposed Warehouse
Lake Forest, California
Project No. 21G135-1
**PLATE C- 4**

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Moisture/Density Relationship
ASTM D-1557




| | |
|---|---|
| Soil ID Number | B-1 @ 0-5' |
| Optimum Moisture (%) | 7.5 |
| Maximum Dry Density (pcf) | 130 |
| Soil Classification | Light Gray to Gray Brown Silty fine Sand |

Proposed Warehouse
Lake Forest, California
Project No. 21G135-1
**PLATE C-5**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# APPENDIX D

# GRADING GUIDE SPECIFICATIONS

These grading guide specifications are intended to provide typical procedures for grading operations. They are intended to supplement the recommendations contained in the geotechnical investigation report for this project. Should the recommendations in the geotechnical investigation report conflict with the grading guide specifications, the more site specific recommendations in the geotechnical investigation report will govern.

## General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.

- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.

- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.

- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.

- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.

- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

## Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.

- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.

- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.

- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.

- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.

- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted

- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

Compacted Fills

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as “contaminated,” and shall be very low to non-expansive with a maximum expansion index (EI) of 50. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.

- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.

- Rock fragments or rocks greater than 12 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. These materials should be placed in accordance with Plate D-8 of these Grading Guide Specifications and in accordance with the following recommendations:

  - Rocks 12 inches or more in diameter should be placed in rows at least 15 feet apart, 15 feet from the edge of the fill, and 10 feet or more below subgrade. Spaces should be left between each rock fragment to provide for placement and compaction of soil around the fragments.

  - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

  - Subsequent rows of rocks should be placed such that they are not directly above a row placed in the previous lift of fill. A minimum 5-foot offset between rows is recommended.

  - To facilitate future trenching, oversized material should not be placed within the range of foundation excavations, future utilities or other underground construction unless specifically approved by the soil engineer and the developer/owner representative.

- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.

- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.

- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.

- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.

- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates D-2, D-4, and D-5.

- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate D-1), as determined by the Geotechnical Engineer.

- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.

- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.

- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.

- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.

- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core

- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.

- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.

- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate D-5).

- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.

- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate D-2).

Cut Slopes

- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.

- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.

- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate D-5.

- Stabilization key excavations should be provided with subdrains. Typical subdrain details are shown on Plates D-6.

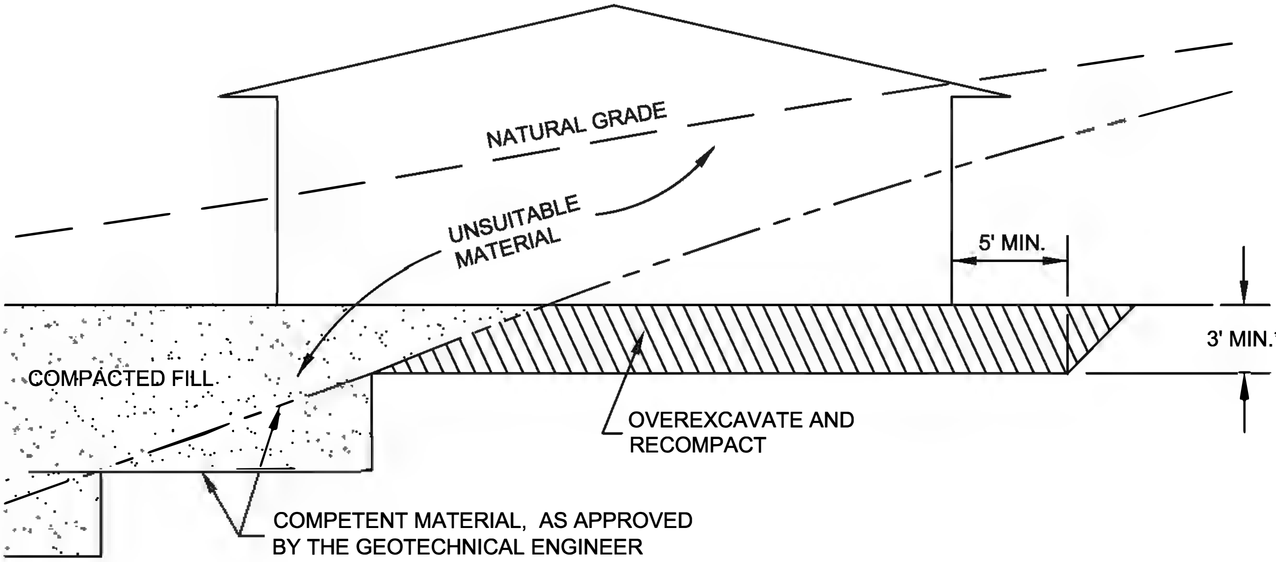
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

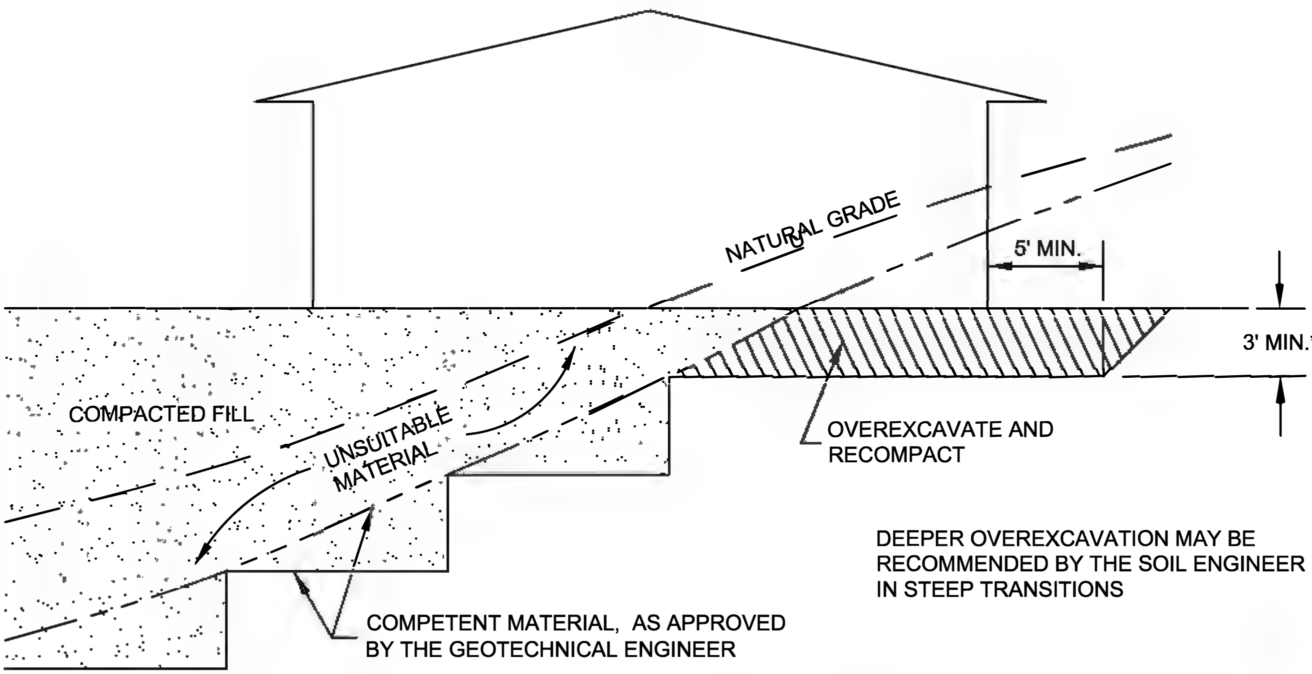
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

## CUT LOT

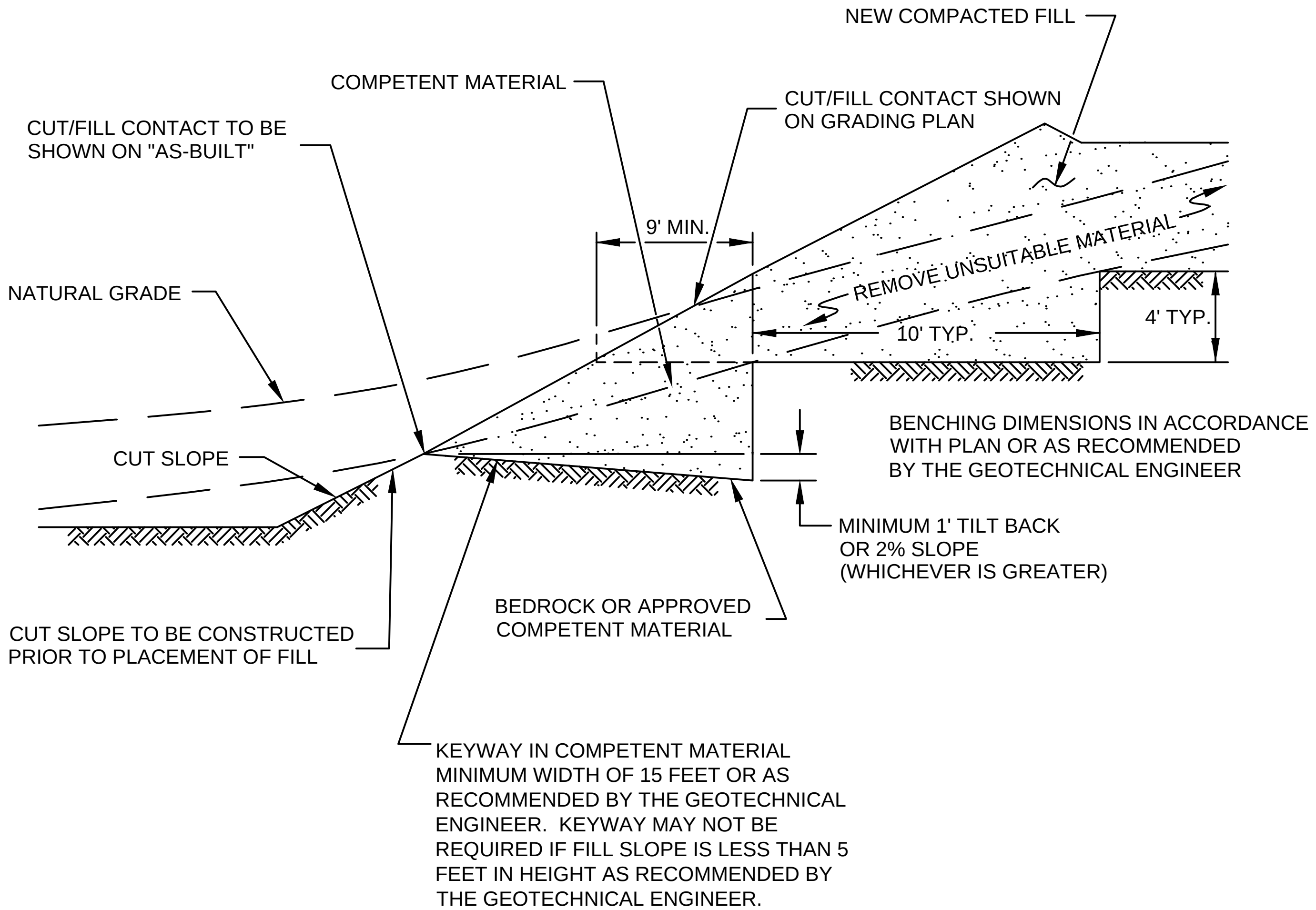



## CUT/FILL LOT (TRANSITION)




*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION.
ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

| | |
|---|---|
| **TRANSITION LOT DETAIL** | |
| GRADING GUIDE SPECIFICATIONS | |
| NOT TO SCALE | SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL |
| DRAWN: JAS<br>CHKD: GKM | |
| **PLATE D-1** | |




**FILL ABOVE CUT SLOPE DETAIL**

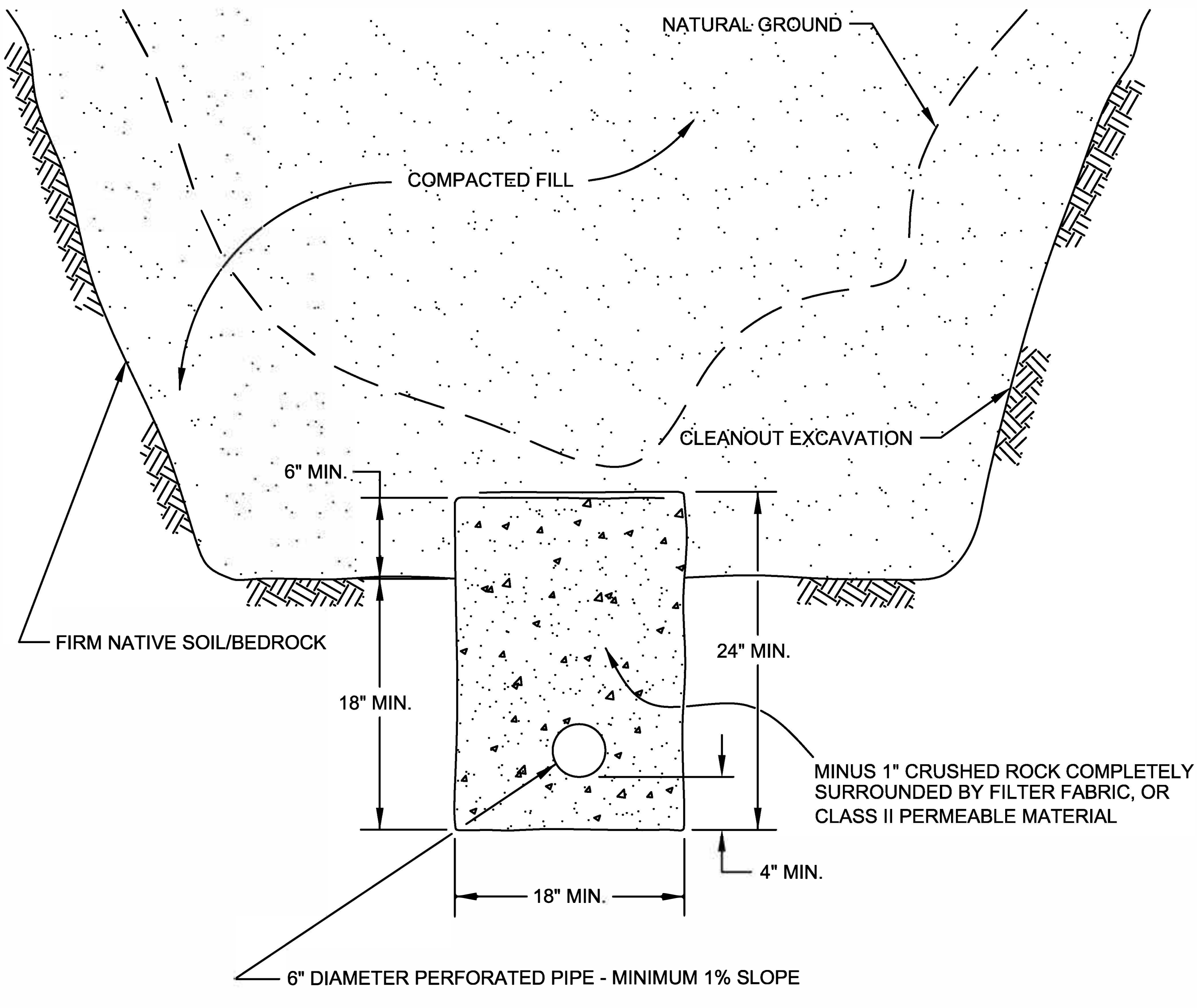
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

**PLATE D-2**

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL




| PIPE MATERIAL | DEPTH OF FILL OVER SUBDRAIN |
|---|---|
| ADS (CORRUGATED POLETHYLENE) | 8 |
| TRANSITE UNDERDRAIN | 20 |
| PVC OR ABS: SDR 35 | 35 |
| SDR 21 | 100 |

**SCHEMATIC ONLY**
**NOT TO SCALE**

**CANYON SUBDRAIN DETAIL**

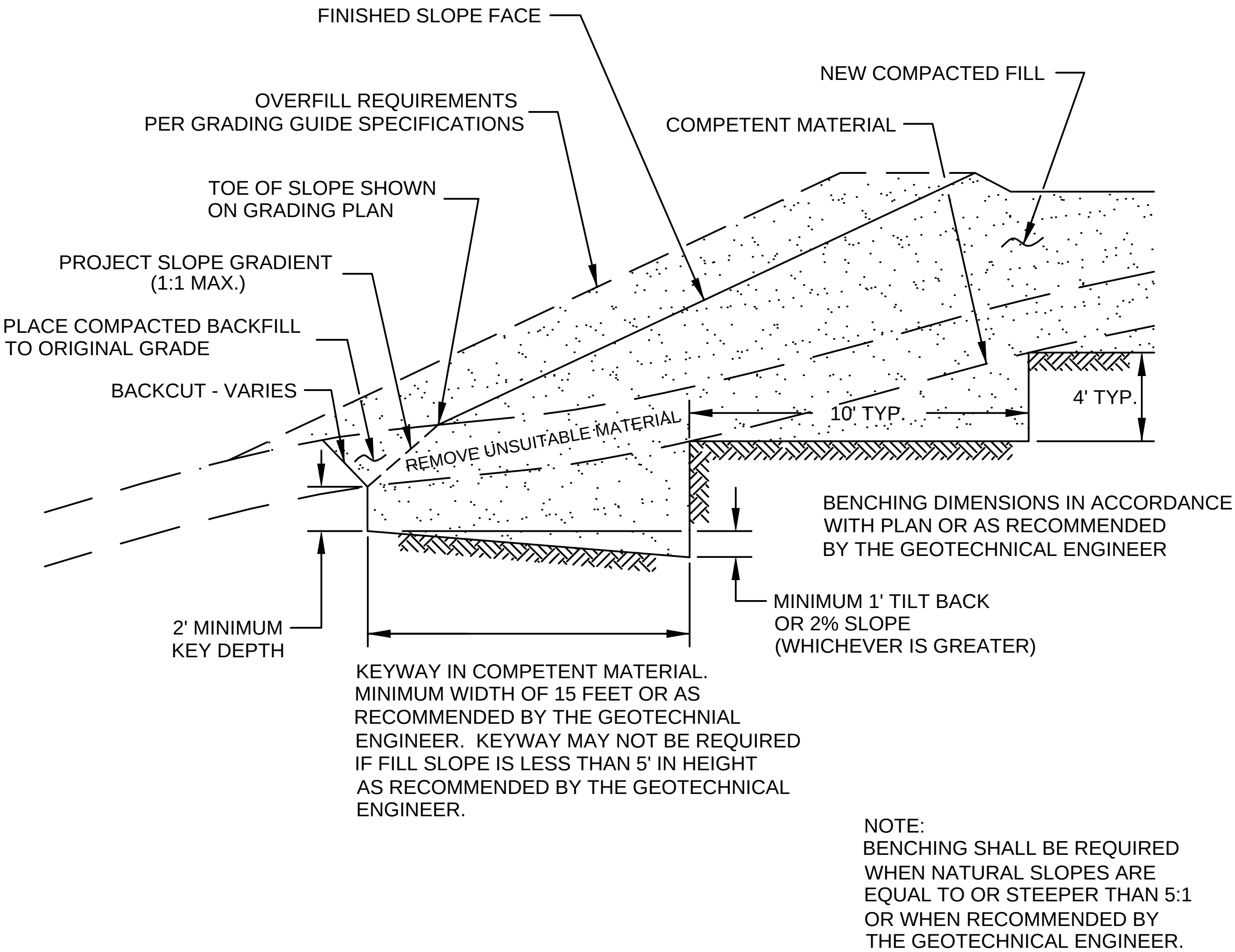
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

**PLATE D-3**

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL




**FILL ABOVE NATURAL SLOPE DETAIL**

GRADING GUIDE SPECIFICATIONS

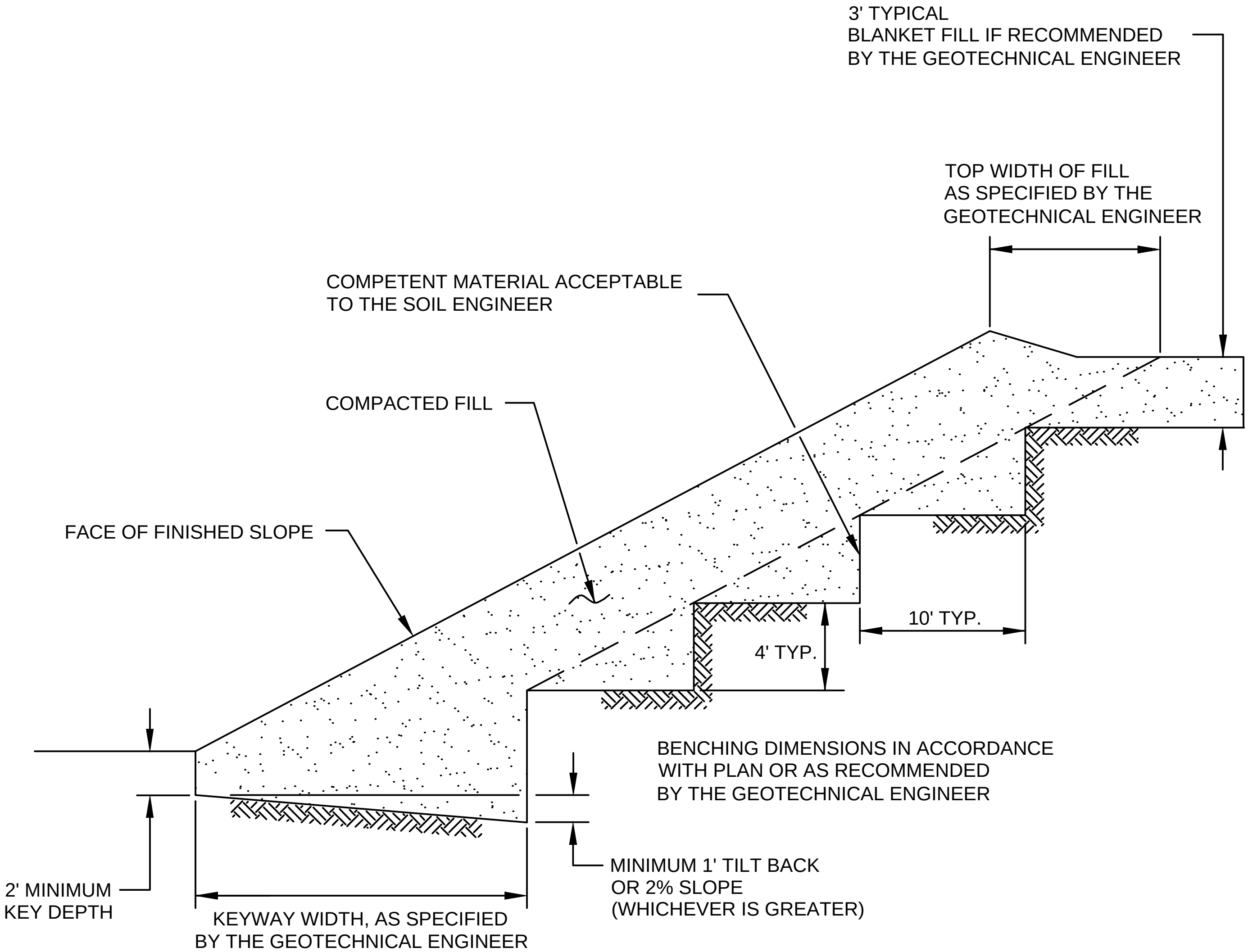
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

**PLATE D-4**

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL




**STABILIZATION FILL DETAIL**

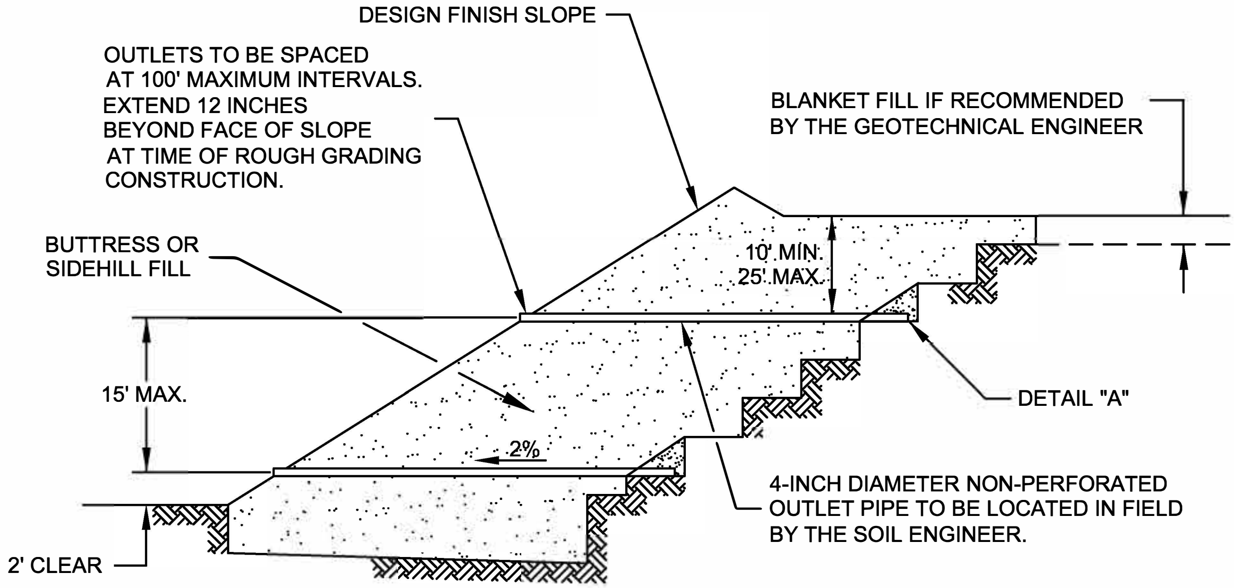
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

**PLATE D-5**

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL




"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

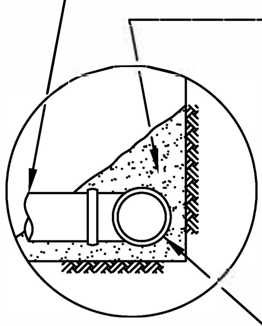
| SIEVE SIZE | PERCENTAGE PASSING |
|---|---|
| 1" | 100 |
| 3/4" | 90-100 |
| 3/8" | 40-100 |
| NO. 4 | 25-40 |
| NO. 8 | 18-33 |
| NO. 30 | 5-15 |
| NO. 50 | 0-7 |
| NO. 200 | 0-3 |

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

| SIEVE SIZE | MAXIMUM PERCENTAGE PASSING |
|---|---|
| 1 1/2" | 100 |
| NO. 4 | 50 |
| NO. 200 | 8 |

SAND EQUIVALENT = MINIMUM OF 50

OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW



FILTER MATERIAL - MINIMUM OF FIVE CUBIC FEET PER FOOT OF PIPE. SEE ABOVE FOR FILTER MATERIAL SPECIFICATION.

ALTERNATIVE: IN LIEU OF FILTER MATERIAL FIVE CUBIC FEET OF GRAVEL PER FOOT OF PIPE MAY BE ENCASED IN FILTER FABRIC. SEE ABOVE FOR GRAVEL SPECIFICATION.

FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

MINIMUM 4-INCH DIAMETER PVC SCH 40 OR ABS CLASS SDR 35 WITH A CRUSHING STRENGTH OF AT LEAST 1,000 POUNDS, WITH A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

DETAIL "A"

NOTES:
1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

**SLOPE FILL SUBDRAINS**

GRADING GUIDE SPECIFICATIONS

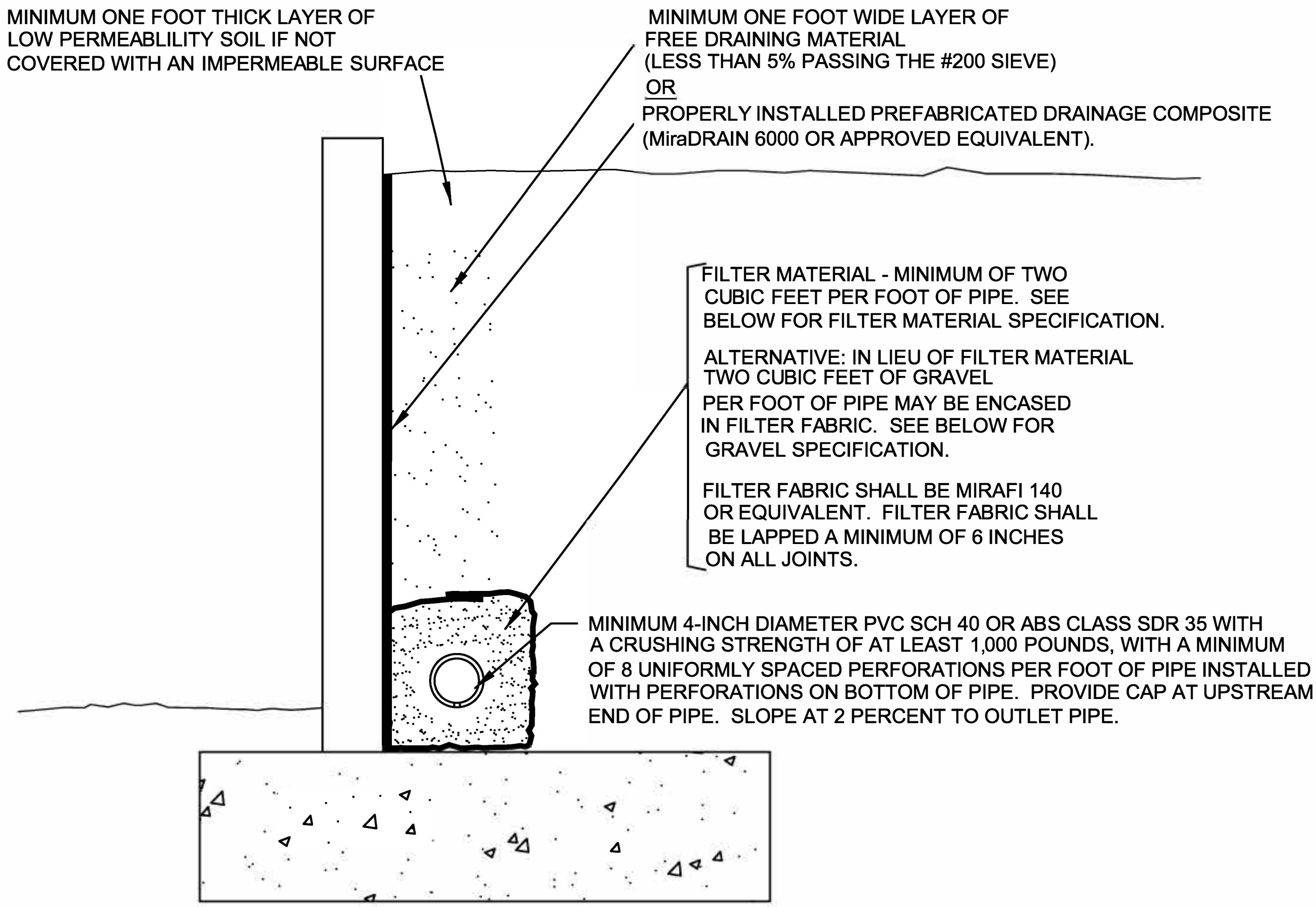
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL




"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

| SIEVE SIZE | PERCENTAGE PASSING |
|---|---|
| 1" | 100 |
| 3/4" | 90-100 |
| 3/8" | 40-100 |
| NO. 4 | 25-40 |
| NO. 8 | 18-33 |
| NO. 30 | 5-15 |
| NO. 50 | 0-7 |
| NO. 200 | 0-3 |

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

| SIEVE SIZE | MAXIMUM PERCENTAGE PASSING |
|---|---|
| 1 1/2" | 100 |
| NO. 4 | 50 |
| NO. 200 | 8 |

SAND EQUIVALENT = MINIMUM OF 50

**RETAINING WALL BACKDRAINS**

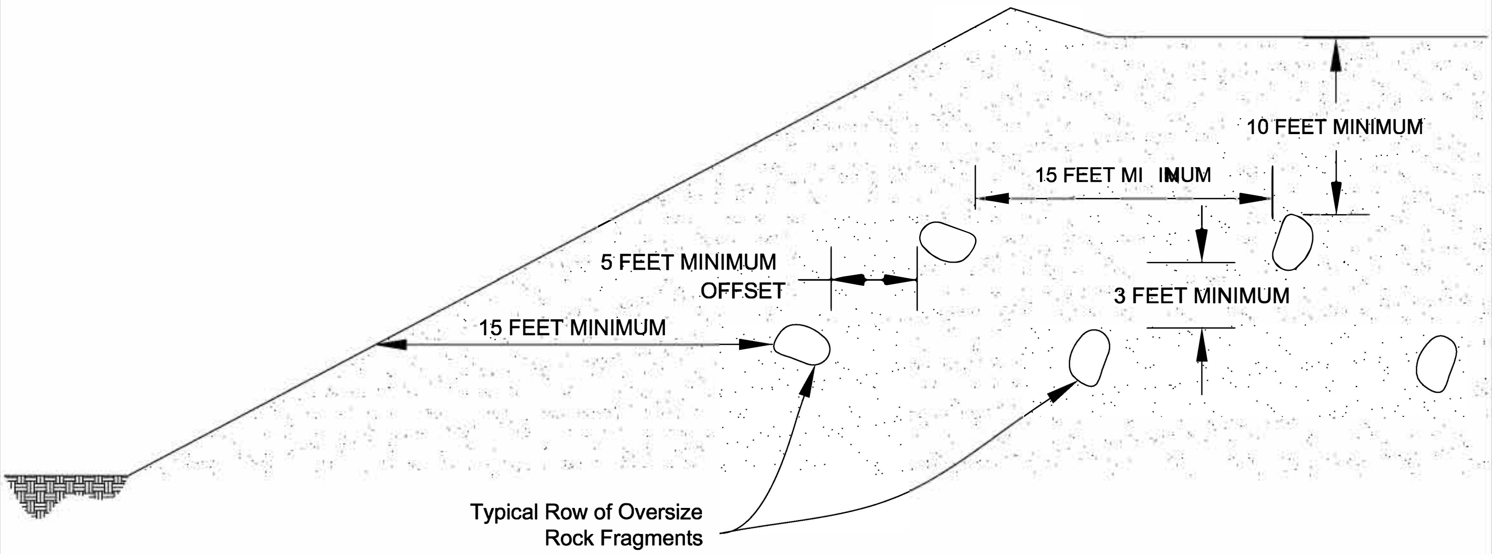
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

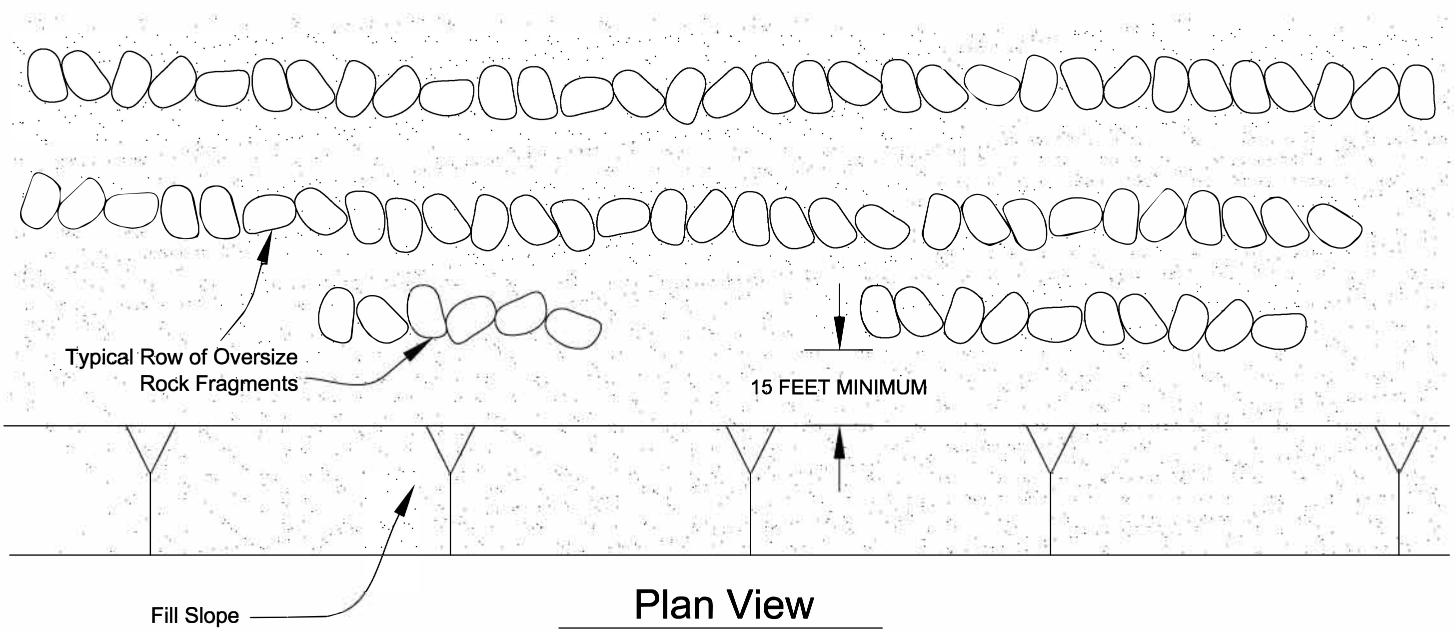
DRAWN: JAS
CHKD: GKM

**PLATE D-7**

SOUTHERN CALIFORNIA GEOTECHNICAL




Section View




Plan View

**PLACEMENT OF OVERSIZED MATERIAL**

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

**PLATE D-8**

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

# APPENDIX E

OSHPD

**Latitude, Longitude: 33.665477, -117.672504**


| | |
|---|---|
| **Date** | 3/24/2021, 1:51:33 PM |
| **Design Code Reference Document** | ASCE7-16 |
| **Risk Category** | III |
| **Site Class** | D - Stiff Soil |

| Type | Value | Description |
|---|---|---|
| $S_S$ | 1.261 | $MCE_R$ ground motion. (for 0.2 second period) |
| $S_1$ | 0.45 | $MCE_R$ ground motion. (for 1.0s period) |
| $S_{MS}$ | 1.261 | Site-modified spectral acceleration value |
| $S_{M1}$ | null -See Section 11.4.8 | Site-modified spectral acceleration value |
| $S_{DS}$ | 0.841 | Numeric seismic design value at 0.2 second SA |
| $S_{D1}$ | null -See Section 11.4.8 | Numeric seismic design value at 1.0 second SA |

| Type | Value | Description |
|---|---|---|
| SDC | null -See Section 11.4.8 | Seismic design category |
| $F_a$ | 1 | Site amplification factor at 0.2 second |
| $F_v$ | null -See Section 11.4.8 | Site amplification factor at 1.0 second |
| PGA | 0.522 | $MCE_G$ peak ground acceleration |
| $F_{PGA}$ | 1.1 | Site amplification factor at PGA |
| $PGA_M$ | 0.575 | Site modified peak ground acceleration |
| $T_L$ | 8 | Long-period transition period in seconds |
| SsRT | 1.261 | Probabilistic risk-targeted ground motion. (0.2 second) |
| SsUH | 1.342 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration |
| SsD | 1.5 | Factored deterministic acceleration value. (0.2 second) |
| S1RT | 0.45 | Probabilistic risk-targeted ground motion. (1.0 second) |
| S1UH | 0.483 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration. |
| S1D | 0.6 | Factored deterministic acceleration value. (1.0 second) |
| PGAd | 0.58 | Factored deterministic acceleration value. (Peak Ground Acceleration) |
| $C_{RS}$ | 0.94 | Mapped value of the risk coefficient at short periods |
| $C_{R1}$ | 0.932 | Mapped value of the risk coefficient at a period of 1 s |

SOURCE: SEAOC/OSHPD Seismic Design Maps Tool
<https://seismicmaps.org/>

**SEISMIC DESIGN PARAMETERS - 2019 CBC**

PROPOSED WAREHOUSE

LAKE FOREST, CALIFORNIA

DRAWN: JAH
CHKD: RGT

SCG PROJECT
21G135-1

**PLATE E-1**

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

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April 8, 2021
(Revised November 18, 2021)

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Black Creek Group
4675 MacArthur Court, Suite 625
Newport Beach, California 92660

Attention: Mr. Chris Sanford
Senior Vice President, Development

Project No.: 21G135-2R2

Subject: Results of Infiltration Testing
Proposed Warehouse
26200 Enterprise Way
Lake Forest, California

Reference: Geotechnical Investigation, Proposed Warehouse, 26200 Enterprise Way, Lake Forest, California, prepared by Southern California Geotechnical (SCG) for Black Creek Group, SCG Project No. 21G135-1R2, dated September 10, 2021.

Dear Mr. Sanford:

In accordance with your request, we have conducted infiltration testing at the subject site. We are pleased to present this report summarizing the results of the infiltration testing and our design recommendations.

## Scope of Services

The scope of services performed for this project was in accordance with our Proposal No. 21P170, dated February 25, 2021. The scope of services included site reconnaissance, subsurface exploration, field testing, and engineering analysis to determine the infiltration rates of the onsite soils. The infiltration testing was performed in general accordance with the guidelines published by Orange County: Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMPs) in South Orange County, Appendix D. These guidelines were most recently updated December 21, 2018.

## Site and Project Description

The site is located at 26200 Enterprise Way in Lake Forest, California. The site is bounded to the north by Enterprise Way and existing commercial/industrial buildings, to the east and south by existing commercial/industrial developments, and to the west by Enterprise Court. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site is an irregular-shaped parcel, 8.83± acres in size. The site is presently developed with one (1) commercial office building, with a first-floor footprint of 75,000± ft$^2$ in size, located in the central area of the site. The development is a two-story building of concrete tilt up construction. A series of Bloomenergy servers, are located on concrete pads along the southern property line, behind the building. The building is surrounded by asphaltic concrete pavements and limited areas of Portland cement concrete pavements. The pavements are in good condition with minor cracking throughout. Landscape planters are present throughout the site and possesses small shrubs, bushes and medium to large trees. The eastern and southern boundaries of the site possess north and west facing slopes. These slopes ascend 10 to 15 feet from the subject site to the adjacent property. A concrete-lined drainage swale is located near the midpoints of the slopes. The remaining areas of the slopes are covered with dense vegetation.

## Proposed Development

A preliminary site plan has been provided to our office by the client. Based on this plan, the site will be developed with one (1) new commercial industrial building, 168,467± $ft^2$ in size, located in the central region of the site. Dock-high doors will be constructed along a portion of the southern building wall. The proposed building is expected to be surrounded by asphaltic concrete pavements in the parking and drive areas, Portland cement concrete pavements in the loading dock areas, and concrete flatwork and landscaped planters throughout the site.

The use of on-site storm water infiltration systems has been proposed for the new development at the site. We understand that the infiltration system may consist of below-grade chambers located in the southeastern corner of the site. The bottom of the infiltration system will be approximately 10± feet below the existing site grades.

## Concurrent Study

SCG recently conducted a geotechnical investigation for this project, referred above. The subsurface exploration for this project consisted of six (6) borings advanced to depths of 20 to 30± feet below the existing site grades. Asphaltic concrete pavement was encountered at the ground surface of all boring locations. The pavement sections generally consisted of 3½ to 4½± inches of asphaltic concrete, underlain by 2½ to 7± inches of aggregate base. Engineered fill soils were encountered at Boring Nos. B-1 and B-4 through B-6, extending to depths of 12 to more than 30± feet. Boring Nos. B-1 and B-5 were terminated in the engineered fill soils. These fill materials consist of medium dense to very dense silty sands with varying clay content. Many samples of the fill soils possess variable coloration and variable strength, indicative of their classification as fill. Capistrano Formation bedrock was encountered beneath the pavements and/or beneath the fill soils at most of the boring locations, extending to at least the maximum depth explored of 30± feet below the existing site grades. The bedrock generally consists of light gray to dark gray fine-grained poorly consolidated silty sandstone. The samples occasionally possess trace amounts of clay, organics and iron oxide staining. Free water was not encountered during the drilling of any of the borings.

## Geologic Conditions

Geologic research indicates that the majority of the site is underlain by white to bluish-white, silty, marine, fine- to medium-grained, thick bedded to massive, poorly sorted arkosic sandstone. mapped as late Miocene to early Pliocene Capistrano Formation, Oso Member (Map Symbol Tco). The Holocene to Pleistocene age Slopewash (Map Symbol Qsw) is mapped in the northwestern and southeastern property lines of the site. The bedding within the Capistrano

Formation is indicated to trend northwest-southeast with a dip of 14 degrees to the north, on the geologic map. The primary available reference applicable to the subject site is the Geologic Map and Sections of the South Half El Toro Quadrangle, Orange County, California, by Donald Fife, 1974. A portion of this map indicating the location of the subject site is included herein as Plate 3 in Appendix A.

Based on the materials encountered in the exploratory borings, the site is underlain by sandstone, and silty sandstone of the Capistrano Formation.

## Subsurface Exploration

### Scope of Exploration

The subsurface exploration conducted for the infiltration testing consisted of two (2) infiltration test borings, advanced to depths of 10± feet below the existing site grades. The infiltration borings were advanced using a conventional truck-mounted drilling rig, equipped with 8-inch-diameter hollow stem augers, and were logged during drilling by a member of our staff. The approximate locations of the infiltration test borings (identified as I-1 and I-2) are indicated on the Infiltration Test Location Plan, enclosed as Plate 2 of this report.

Upon the completion of the infiltration borings, the bottom of each test boring was covered with 2± inches of clean ¾-inch gravel. A sufficient length of 3-inch-diameter perforated PVC casing was then placed into each test hole so that the PVC casing extended from the bottom of the test hole to the ground surface. Clean ¾-inch gravel was then installed in the annulus surrounding the PVC casing.

## Geotechnical Conditions

### Pavements

Asphaltic Concrete (AC) pavements were encountered at the ground surface of both infiltration test locations. The pavements consisted of 2½ to 3½± inches of AC with 5± inches of Aggregate Base (AB) at each location.

### Engineered Fill (Afe)

Engineered fill soils were encountered beneath the pavements at both infiltration test locations, extending to at least the maximum depth explored of 10± feet below ground surface. These fill materials consist of medium dense to dense silty fine sands and clayey sands. Some samples of the fill soils possess variable coloration and variable strength, indicative of their classification as fill.

## Infiltration Testing

As previously mentioned, the infiltration testing was performed in general accordance with the Orange County guidelines: Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMPs) in South Orange County, Appendix D.

Pre-soaking

In accordance with the county infiltration standards, all of the infiltration test borings were pre-soaked prior to the infiltration testing. The pre-soaking process consisted of filling the test borings by inverting a full 5-gallon bottle of clear water supported over each hole so that the water level reaches a level of at least 5 times the hole's radius above the gravel at the bottom of each hole. The pre-soaking was completed after all of the water had percolated through each test hole or after 15 hours since initiating the pre-soak.

Infiltration Testing

Following the pre-soaking process of the infiltration test borings, SCG performed the infiltration testing. Each test hole was filled with water to a depth of at least 5 times the hole's radius above the gravel at the bottom of each test hole, and less than or equal to the water level used during the pre-soaking process. In accordance with the Orange County guidelines, since "non-sandy soils" were encountered at the bottom of both infiltration borings, readings were taken at 30-minute intervals for a total of 6 hours. After each reading, the borings were refilled to the correct water level above the gravel at the bottom of each test hole. The water level readings are presented on the spreadsheets enclosed with this report. The infiltration rates for each of the timed intervals are also tabulated on the spreadsheets.

The infiltration rates from the test are tabulated in inches per hour. In accordance with the typically accepted practice, it is recommended that the most conservative reading from the latter part of the infiltration tests be used as the design infiltration rate. The rates are summarized below:

| Infiltration Test No. | Depth (feet) | Soil Description | Infiltration Rate (inches/hour) |
|---|---|---|---|
| I-1 | 10 | ENGINEERED FILL: Light Gray to Brown Silty fine Sand | 0.1 |
| I-2 | 10 | ENGINEERED FILL: Gray Brown Clayey fine Sand | 0.3 |

## Laboratory Testing

Moisture Content

The moisture contents for the recovered soil samples within the borings were determined in accordance with ASTM D-2216 and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Grain Size Analysis

The grain size distribution of selected soils collected from the bottom of each infiltration test boring have been determined using a range of wire mesh screens. These tests were performed in general accordance with ASTM D-422 and/or ASTM D-1140. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of these tests are presented on Plates C-1 through C-2 of this report.

## Design Recommendations

Two (2) infiltration tests were performed at the subject site. As noted above, the infiltration rates at these locations range from 0.1 to 0.3 inches per hour.

Based on the results of the infiltration testing at the subject site, infiltration is not considered feasible at this site due to the presence of dense engineered fill soils, comprised of silty sands and clayey sands, which possess very poor infiltration characteristics.

## General Comments

This report has been prepared as an instrument of service for use by the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, structural engineer, and/or civil engineer. The design of the infiltration system is the responsibility of the civil engineer. The role of the geotechnical engineer is limited to determination of infiltration rate only. By using the design infiltration rates contained herein, the civil engineer agrees to indemnify, defend, and hold harmless the geotechnical engineer for all aspects of the design and performance of the infiltration system. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between trench locations and testing depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted. The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

## Closure

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

**SOUTHERN CALIFORNIA GEOTECHNICAL, INC.**

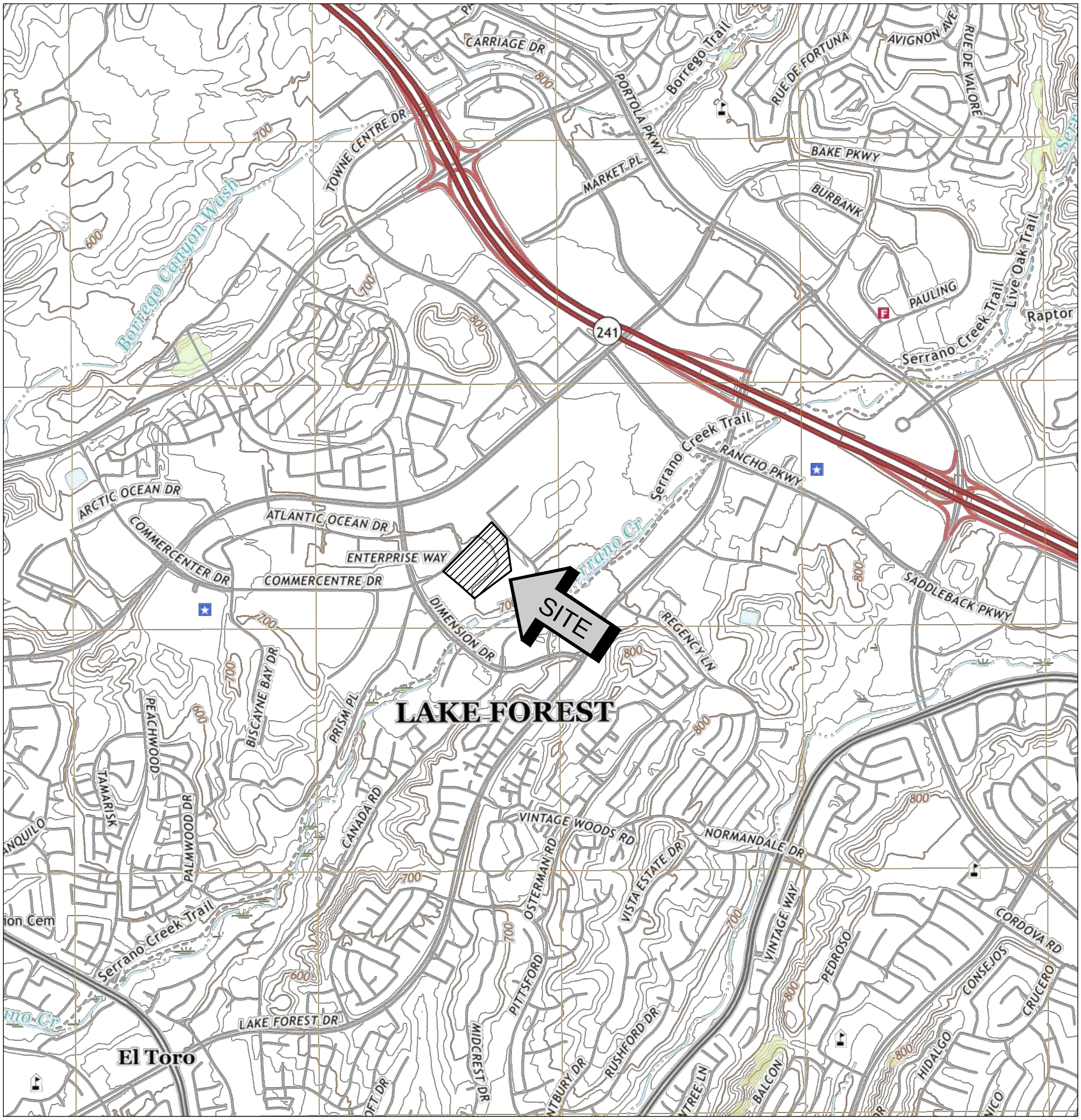
Jose Zuniga
Staff Engineer

Gregory K. Mitchell, GE 2364
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
GREGORY K. MITCHELL
No. 2364
GEOTECHNICAL
STATE OF CALIFORNIA

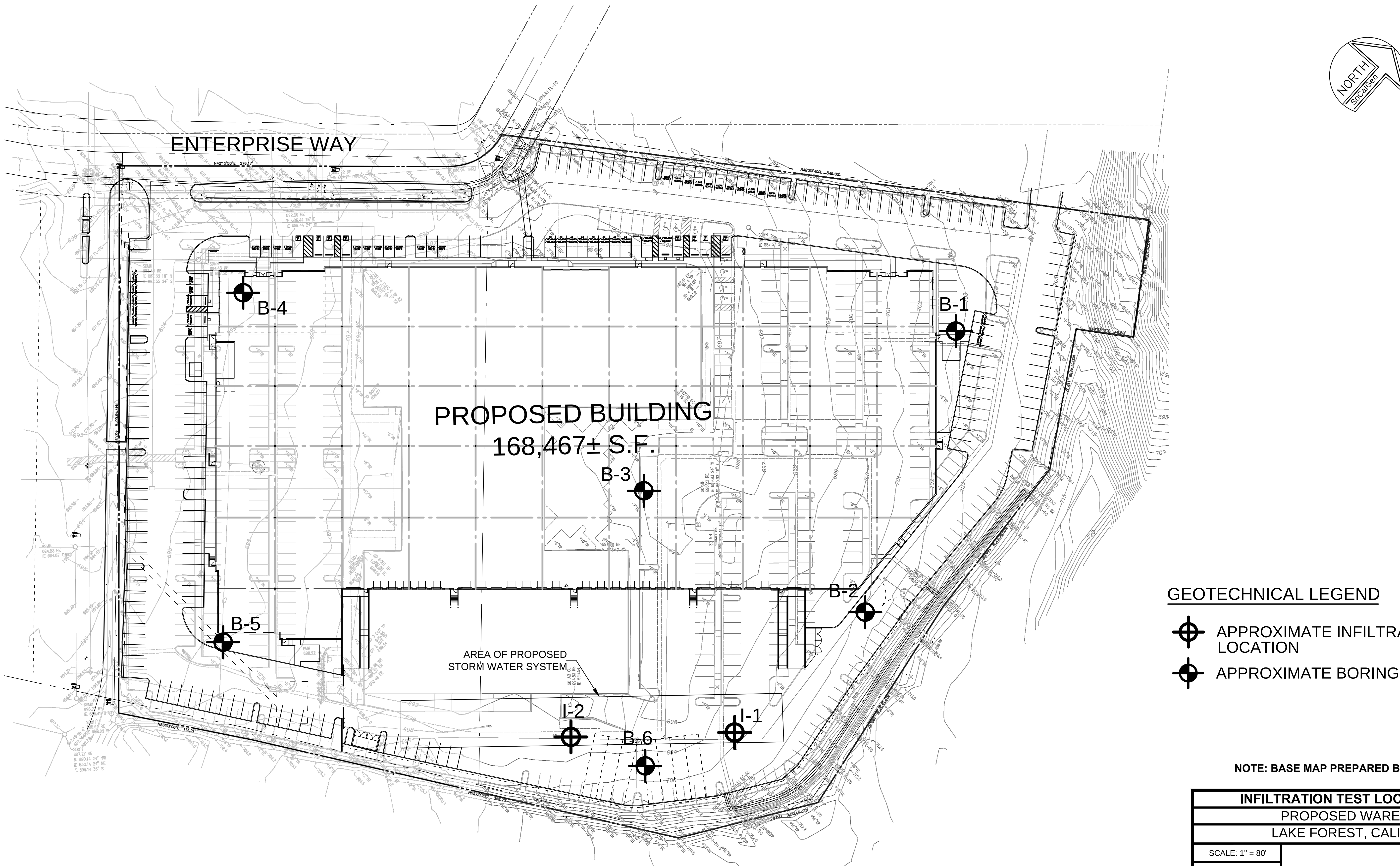
Distribution: (1) Addressee

Enclosures:
- Plate 1 - Site Location Map
- Plate 2 - Infiltration Test Location Plan
- Plate 3 – Geologic Map
- Boring Log Legend and Logs (4 pages)
- Infiltration Test Results Spreadsheets (2 pages)
- Grain Size Distribution Graphs (2 pages)




SOURCE: USGS TOPOGRAPHIC MAP OF LAKE FOREST QUADRANGLE, ORANGE COUNTY, CALIFORNIA, 2018

| SITE LOCATION MAP | |
|---|---|
| PROPOSED WAREHOUSE | |
| LAKE FOREST, CALIFORNIA | |
| SCALE: 1" = 2000' | SOUTHERN CALIFORNIA GEOTECHNICAL |
| DRAWN: MD<br>CHKD: RGT | |
| SCG PROJECT<br>21G135-2 | |
| **PLATE 1** | |




GEOTECHNICAL LEGEND

- APPROXIMATE INFILTRATION TEST LOCATION
- APPROXIMATE BORING LOCATION

NOTE: BASE MAP PREPARED BY KIER + WRIGHT

| | |
|---|---|
| **INFILTRATION TEST LOCATION PLAN** | |
| PROPOSED WAREHOUSE | |
| LAKE FOREST, CALIFORNIA | |
| SCALE: 1" = 80' | |
| DRAWN: JAH<br>CHKD: RGT | |
| SCG PROJECT<br>21G135-2R | |
| **PLATE 2** | |


LEGEND

Qsw

SLOPEWASH. (Holocene to Pleistocene(?))

Lithology: Varies with parent materials. In general consists of light olive-gray to olive-gray, silty to clayey, organicly rich soil which is plastic when wet and hard when dry. Consists of detrital materials resulting from debris flows or creep aided by rill wash, ground water, and unchanneled surficial drainage. Accumulates near the base of slopes and at the mouths of small canyons. Includes colluvium and minor residu soils. It is commonly transitional with adjacent soils or alluvial deposits. Mapped where estimated to exceed three feet in thickness.

Tco

CAPISTRANO FORMATION, OSO MEMBER. (late Miocene to early Pliocene)

Lithology: Marine, white to bluish-white, silty, fine-to medium-grained, thick bedded to massive, poorly sorted arkosic sandstone. Commonly cross-bedded. Locally contains calcareous concretions as much as two feet in diameter. Concretionary zones commonly are found in association with marine vertebrate fossils, but locally friable sand lenses are encountered. Bentonite beds, 1 to 2 inches thick, were observed in an anticline on the east flank of Aliso Creek.

SOURCE: "GEOLOGY MAP OF THE SOUTH HALF OF THE EL TORO QUADRANGLE, ORANGE COUNTY, CALIFORNIA" BY DONALD L. FIFE, 1974

**GEOLOGIC MAP**

PROPOSED WAREHOUSE

LAKE FOREST, CALIFORNIA

SCALE: 1" = 2000'

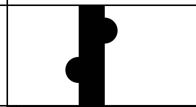
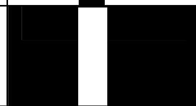
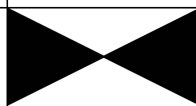
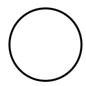
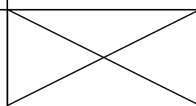
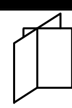
DRAWN: JAH
CHKD: GKM

SCG PROJECT
21G135-2

**PLATE 3**

SOUTHERN CALIFORNIA GEOTECHNICAL

# BORING LOG LEGEND

| SAMPLE TYPE | GRAPHICAL SYMBOL | SAMPLE DESCRIPTION |
|---|---|---|
| AUGER |  | SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED) |
| CORE |  | ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK. |
| GRAB |  | SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED) |
| CS |  | CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED) |
| NSR |  | NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL. |
| SPT |  | STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED) |
| SH |  | SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED) |
| VANE |  | VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED. |

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

BLOW COUNT: Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.

POCKET PEN.: Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.

GRAPHIC LOG: Graphic Soil Symbol as depicted on the following page.

DRY DENSITY: Dry density of an undisturbed or relatively undisturbed sample in $lbs/ft^3$.

MOISTURE CONTENT: Moisture content of a soil sample, expressed as a percentage of the dry weight.

LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

PLASTIC LIMIT: The moisture content above which a soil behaves as a plastic.

PASSING #200 SIEVE: The percentage of the sample finer than the #200 standard sieve.

UNCONFINED SHEAR: The shear strength of a cohesive soil sample, as measured in the unconfined state.

# SOIL CLASSIFICATION CHART

| MAJOR DIVISIONS | | | SYMBOLS GRAPH | SYMBOLS LETTER | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|---|
| COARSE GRAINED SOILS<br>MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE | GRAVEL AND GRAVELLY SOILS<br>MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE | CLEAN GRAVELS<br>(LITTLE OR NO FINES) | | GW | WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | | | | GP | POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | | GRAVELS WITH FINES<br>(APPRECIABLE AMOUNT OF FINES) | | GM | SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES |
| | | | | GC | CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES |
| | SAND AND SANDY SOILS<br>MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE | CLEAN SANDS<br>(LITTLE OR NO FINES) | | SW | WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES |
| | | | | SP | POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES |
| | | SANDS WITH FINES<br>(APPRECIABLE AMOUNT OF FINES) | | SM | SILTY SANDS, SAND - SILT MIXTURES |
| | | | | SC | CLAYEY SANDS, SAND - CLAY MIXTURES |
| FINE GRAINED SOILS<br>MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE | SILTS AND CLAYS | LIQUID LIMIT LESS THAN 50 | | ML | INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY |
| | | | | CL | INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS |
| | | | | OL | ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY |
| | SILTS AND CLAYS | LIQUID LIMIT GREATER THAN 50 | | MH | INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS |
| | | | | CH | INORGANIC CLAYS OF HIGH PLASTICITY |
| | | | | OH | ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS |
| HIGHLY ORGANIC SOILS | | | | PT | PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS |

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
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# BORING NO. I-1

JOB NO.: 21G135-2
PROJECT: Proposed Warehouse
LOCATION: Lake Forest, California

DRILLING DATE: 3/12/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: ---
READING TAKEN: At Completion

| FIELD RESULTS | | | | | | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION<br>SURFACE ELEVATION: 689.5 feet MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | 3½± Inches Asphaltic Concrete, 5± Inches Aggregate Base | | | | | | | |
| | X | 31 | | | ENGINEERED FILL (Afe): Light Gray Silty fine Sand, trace Iron oxide staining, dense-moist | | 12 | | | | | |
| | | | | | @ 3 feet, moist to very moist | | | | | | | |
| | X | 35 | | | | | 9 | | | | | |
| 5 | | | | | | | | | | | | |
| | X | 39 | | | | | 20 | | | | | |
| | | | | | ENIGEERED FILL (Afe): Light Gray to Brown Silty fine Sand, dense-moist to very moist | | 10 | | | | | |
| | X | 36 | | | | | 15 | | | 27 | | |
| 10 | | | | | | | | | | | | |

Boring Terminated at 10'

TBL 21G135-2.GPJ SOCALGEO.GDT 4/9/21

**TEST BORING LOG** **PLATE B-1**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. I-2

JOB NO.: 21G135-2
PROJECT: Proposed Warehouse
LOCATION: Lake Forest, California

DRILLING DATE: 3/12/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: ---
READING TAKEN: At Completion

SURFACE ELEVATION: 670 feet MSL

| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 2½± Inches Asphaltic Concrete, 5± Inches Aggregate Base | | | | | | | |
| | X | 38 | | | ENIGEERED FILL (Afe): Light Gray Silty fine Sand, trace Iron oxide staining, dense-moist | | 8 | | | | | |
| | | | | | @ 3 feet, moist to very moist | | | | | | | |
| | X | 40 | | | | | 11 | | | | | |
| 5 | | | | | | | | | | | | |
| | X | 32 | | | | | 10 | | | | | |
| | X | 16 | | | ENIGEERED FILL (Afe): Gray Brown Clayey fine Sand, trace Silt, medium dense-moist | | 9 | | | 29 | | |
| 10 | | | | | | | | | | | | |

Boring Terminated at 10'

TBL 21G135-2.GPJ SOCALGEO.GDT 4/9/21

**TEST BORING LOG**

**PLATE B-2**

**INFILTRATION CALCULATIONS**

| | |
|---|---|
| Project Name | Proposed Warehouse |
| Project Location | Lake Forest, California |
| Project Number | 21G135-2 |
| Engineer | Oscar Sandoval |

| | | |
|---|---|---|
| Test Hole Radius | 4 | (in) |
| Test Depth | 10.2 | (ft) |

| | |
|---|---|
| Infiltration Test Hole | I-1 |

| Interval Number | | Time | Time Interval (min) | Water Depth (ft) | Change in Water Level (ft) | Average Head Height (ft) | Infiltration Rate Q (in/hr) |
|---|---|---|---|---|---|---|---|
| 1 | Initial | 8:30 AM | 30.0 | 8.20 | 0.15 | 1.93 | 0.29 |
| | Final | 9:00 AM | | 8.35 | | | |
| 2 | Initial | 9:02 AM | 30.0 | 8.20 | 0.20 | 1.90 | 0.39 |
| | Final | 9:32 AM | | 8.40 | | | |
| 3 | Initial | 9:34 AM | 30.0 | 8.20 | 0.20 | 1.90 | 0.39 |
| | Final | 10:04 AM | | 8.40 | | | |
| 4 | Initial | 10:04 AM | 30.0 | 8.20 | 0.20 | 1.90 | 0.39 |
| | Final | 10:34 AM | | 8.40 | | | |
| 5 | Initial | 10:36 AM | 30.0 | 8.20 | 0.16 | 1.92 | 0.31 |
| | Final | 11:06 AM | | 8.36 | | | |
| 6 | Initial | 11:08 AM | 30.0 | 8.20 | 0.16 | 1.92 | 0.31 |
| | Final | 11:38 AM | | 8.36 | | | |
| 7 | Initial | 11:40 AM | 30.0 | 8.20 | 0.16 | 1.92 | 0.31 |
| | Final | 12:10 PM | | 8.36 | | | |
| 8 | Initial | 12:13 PM | 30.0 | 8.20 | 0.16 | 1.92 | 0.31 |
| | Final | 12:43 PM | | 8.36 | | | |
| 9 | Initial | 12:45 PM | 30.0 | 8.20 | 0.15 | 1.93 | 0.29 |
| | Final | 1:15 PM | | 8.35 | | | |
| 10 | Initial | 1:20 PM | 30.0 | 8.20 | 0.15 | 1.93 | 0.29 |
| | Final | 1:50 PM | | 8.35 | | | |
| 11 | Initial | 1:52 PM | 30.0 | 8.20 | 0.15 | 1.93 | 0.29 |
| | Final | 2:22 PM | | 8.35 | | | |
| 12 | Initial | 2:26 PM | 30.0 | 8.20 | 0.15 | 1.93 | 0.29 |
| | Final | 2:56 PM | | 8.35 | | | |

Per County Standards, Infiltration Rate calculated as follows:

$$\boxed{Q = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}}$$

Where: $Q$ = Infiltration Rate (in inches per hour)

$\Delta H$ = Change in Height (Water Level) over the time interval

$r$ = Test Hole (Borehole) Radius

$\Delta t$ = Time Interval

$H_{avg}$ = Average Head Height over the time interval

## INFILTRATION CALCULATIONS

| | |
|---|---|
| Project Name | Proposed Warehouse |
| Project Location | Lake Forest, California |
| Project Number | 21G135-2 |
| Engineer | Oscar Sandoval |

| | | |
|---|---|---|
| Test Hole Radius | 4 | (in) |
| Test Depth | 10.2 | (ft) |

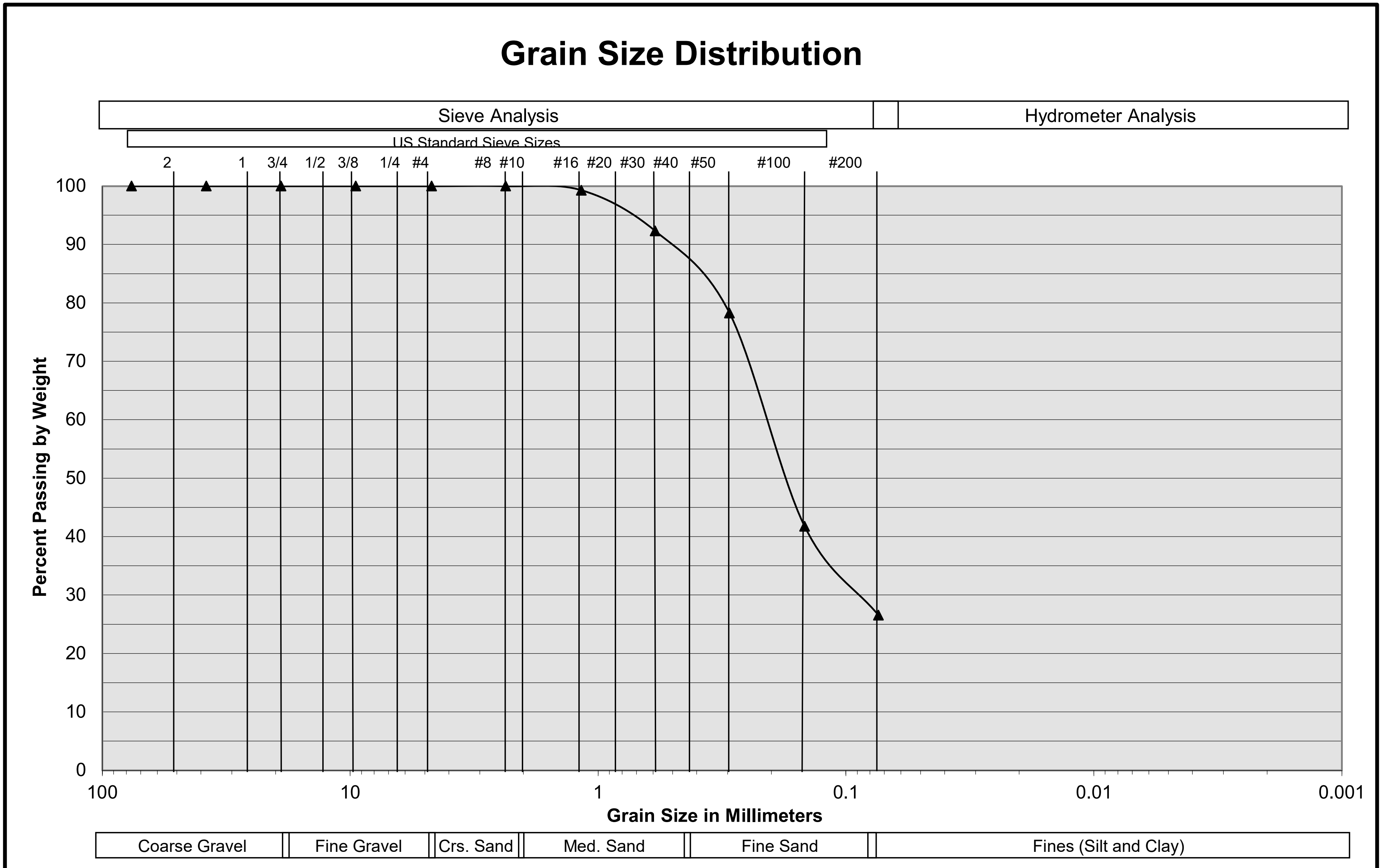
| | |
|---|---|
| Infiltration Test Hole | I-1 |

| Interval Number | | Time | Time Interval (min) | Water Depth (ft) | Change in Water Level (ft) | Average Head Height (ft) | Infiltration Rate Q (in/hr) |
|---|---|---|---|---|---|---|---|
| 1 | Initial | 7:55 AM | 30.0 | 8.20 | 0.05 | 1.98 | 0.09 |
| | Final | 8:25 AM | | 8.25 | | | |
| 2 | Initial | 8:25 AM | 30.0 | 8.20 | 0.05 | 1.98 | 0.09 |
| | Final | 8:55 AM | | 8.25 | | | |
| 3 | Initial | 8:55 AM | 30.0 | 8.20 | 0.07 | 1.97 | 0.13 |
| | Final | 9:25 AM | | 8.27 | | | |
| 4 | Initial | 9:25 AM | 30.0 | 8.20 | 0.07 | 1.97 | 0.13 |
| | Final | 9:55 AM | | 8.27 | | | |
| 5 | Initial | 9:55 AM | 30.0 | 8.20 | 0.06 | 1.97 | 0.11 |
| | Final | 10:25 AM | | 8.26 | | | |
| 6 | Initial | 10:25 AM | 30.0 | 8.20 | 0.06 | 1.97 | 0.11 |
| | Final | 10:55 AM | | 8.26 | | | |
| 7 | Initial | 10:55 AM | 30.0 | 8.20 | 0.06 | 1.97 | 0.11 |
| | Final | 11:25 AM | | 8.26 | | | |
| 8 | Initial | 11:25 AM | 30.0 | 8.20 | 0.06 | 1.97 | 0.11 |
| | Final | 11:55 AM | | 8.26 | | | |
| 9 | Initial | 11:55 AM | 30.0 | 8.20 | 0.06 | 1.97 | 0.11 |
| | Final | 12:25 PM | | 8.26 | | | |
| 10 | Initial | 12:25 PM | 30.0 | 8.20 | 0.05 | 1.98 | 0.09 |
| | Final | 12:55 PM | | 8.25 | | | |
| 11 | Initial | 12:55 PM | 30.0 | 8.20 | 0.05 | 1.98 | 0.09 |
| | Final | 1:25 PM | | 8.25 | | | |
| 12 | Initial | 1:25 PM | 30.0 | 8.20 | 0.05 | 1.98 | 0.09 |
| | Final | 1:55 PM | | 8.25 | | | |

Per County Standards, Infiltration Rate calculated as follows:

$$Q = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Where:
- $Q$ = Infiltration Rate (in inches per hour)
- $\Delta H$ = Change in Height (Water Level) over the time interval
- $r$ = Test Hole (Borehole) Radius
- $\Delta t$ = Time Interval
- $H_{avg}$ = Average Head Height over the time interval




| | |
|---|---|
| Sample Description | I-1 @ 8.5' |
| Soil Classification | Light Gray to Brown Silty fine Sand |

Proposed Warehouse
Lake Forest, California
Project No. 21G135-2
**PLATE C- 1**

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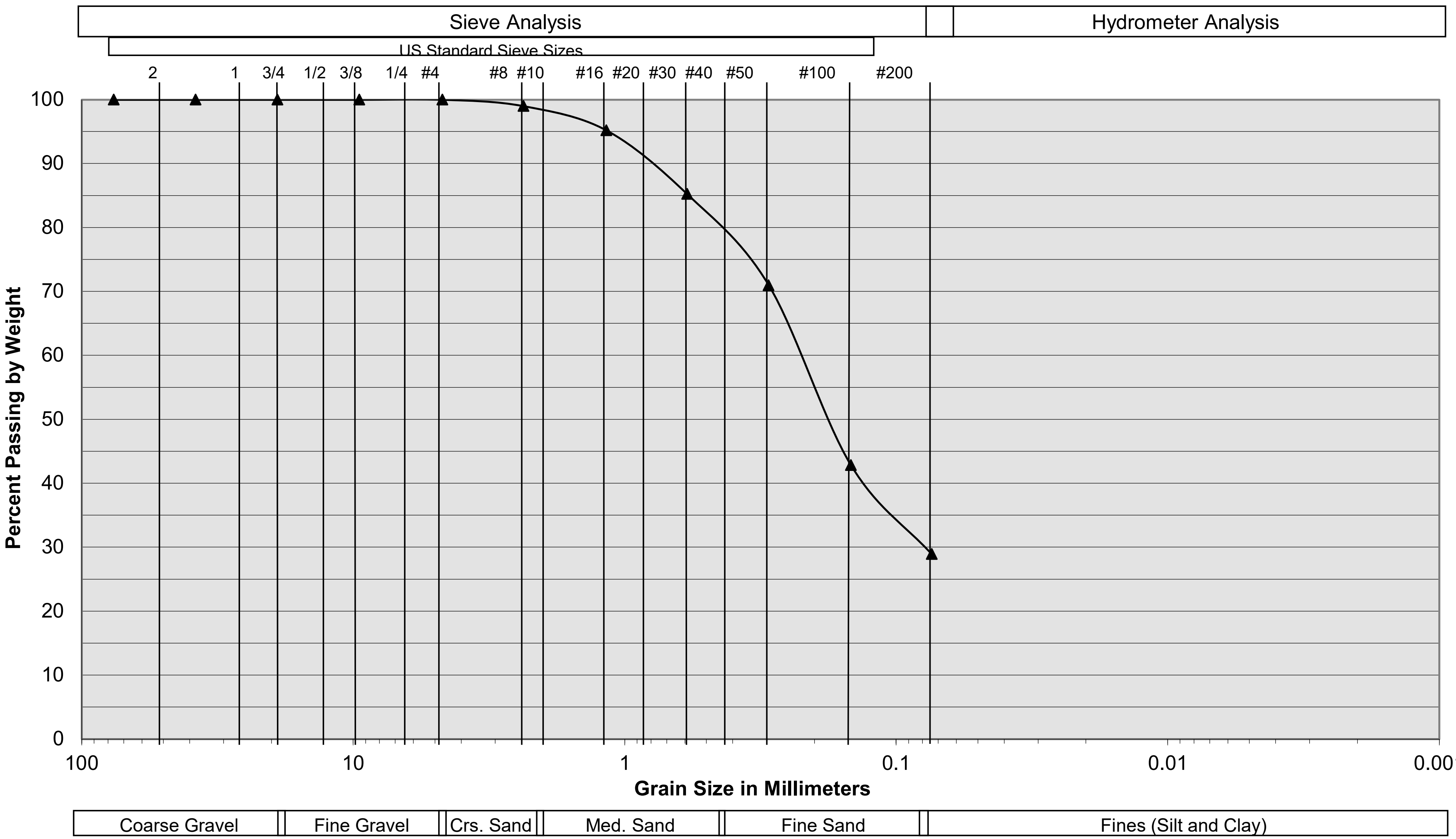
# Grain Size Distribution




| | |
|---|---|
| Sample Description | I-2 @ 8.5' |
| Soil Classification | Gray Brown Clayey fine Sand |

Proposed Warehouse
Lake Forest, California
Project No. 21G135-2
**PLATE C- 2**

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